INSTITUTE
OF
HYDROLOGY

METHODS OF FLOOD ESTIMATION :

A GUIDE TO THE FLOOD STUDIES REPORT

by

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CONTENTS

1 INTRODUCTION

The results of an extensive research programme on flood prediction techniques were published as the 'Flood Studies Report[1]' in 1975. This research, requested originally by the Institution of Civil Engineers, produced improved methods of flood estimation for use in engineering design, the culmination of collaboration between the Institute of Hydrology, the Meteorological Office, the Hydraulics Research Station, the Irish Office of Public Works, and several other organisations.

The Flood Studies Report is necessarily a massive document. The five volumes contain not only the estimation techniques proposed but also the arguments and analyses on which the recommendations are based, together with the original data used. This short guide has been compiled to help the engineer to apply the suggested methods of flood estimation without having to refer continually to formulae and tables from a large set of books. Some extra material is included based on comments and discussions since the original report was published; an example of this is a simple formula for the estimated maximum flood.

The guide does not try to be self-contained - most users will want to study the main report in depth at some stage - but merely presents a more succinct set of rules and examples in a convenient form. Further, while portions of maps or reduced maps have been reproduced to explain the procedures, accurate use of the methods will require reference to the full set of maps contained in the Report.

A summary of the Report is included in an appendix to give those readers who have not been able to study the main report some knowledge of its contents before they turn to examples; this summary defines and explains some of the terms used in the guide.

References to the Flood Studies Report are given by volume (in Roman numerals), chapter and section, e.g. I.6.8.4, or by page, e.g. I.475. Where tables or diagrams from the report have been reproduced, their numbering by chapter has been retained; where a new example has been illustrated, this has been indicated, for instance, by amending Figure 6.66 to Figure 6.66A.

The guide, like the report, is confined to hydrological estimation. Advice to the design engineer on the risk of exceedance of a design flood which it is appropriate to accept in given circumstances is the proper concern of other professional manuals or of legislation. Where the statistician and hydrologist can give guidance is in tracing the relationship between the period during which a structure is at risk, or the design life, the risk of exceedance during that design life, and the return period to be assigned to the design flood.

The reader will find a number of points at which choice of method has to be made; it is hoped that this guide provides the framework for this choice. The design engineer must be allowed to exercise judgement, for instance in using records from adjacent rivers.

[1] The *Flood Studies Report* published by the Natural Environment Research Council consists of five volumes: Vol I (570p) - Hydrological studies; Vol II (91p) - Meteorological studies; Vol III (85p) - Flood routing studies; Vol IV (549p) - Hydrological data; Vol V - Maps (24).

2 CHOICE OF METHODS OF FLOOD ESTIMATION

There are no rigid rules to provide the design engineer with a specification of the design flood. He will have to determine this for each project according to policy or a code of practice, or following an economic analysis. The risk which is accepted of the design flood being exceeded during the life of the project determines the return period of the design flood. They are related by:

$$r = 1 - (1-1/T)^{L}$$

where r is the risk and L and T are respectively the life of the project and the return period in years. If a policy decision is made that no appreciable risk of failure can be accepted, then an estimate of the maximum flood is required. It may also be necessary to allow for errors of estimate of the design flood due to uncertainty in choice of model or of its parameters.

There are a number of routes for estimating the design flood and some have advantages over others in certain circumstances. It is not always possible, or even desirable, to eliminate alternative methods of estimation, so that estimates may be compared and combined.

The two main routes are:

(a) through statistical analysis of peak flows, and

(b) through unit hydrograph synthesis of the flood corresponding to a design storm.

The choice between these two depends largely on the answers to two questions:

Is the hydrograph, or the detailed shape of the flood, required in addition to the instantaneous peak flow (as for example when the flood needs routing through a reservoir)?

Is an estimate of the maximum flood required, rather than an estimate of the flood of a given frequency or return period?

If the answer to either question is yes, then unit hydrograph methods are necessary. If neither the hydrograph nor the maximum flood is needed, then a choice of methods is available, according to whether there are long, short, or no records available for the design site or nearby on the same river.

These choices are summarised in the flow diagram opposite. (Note that this diagram is also included as a 'fold-out' insert between pages 36 and 37 for ease of reference when reading later sections of the guide.)

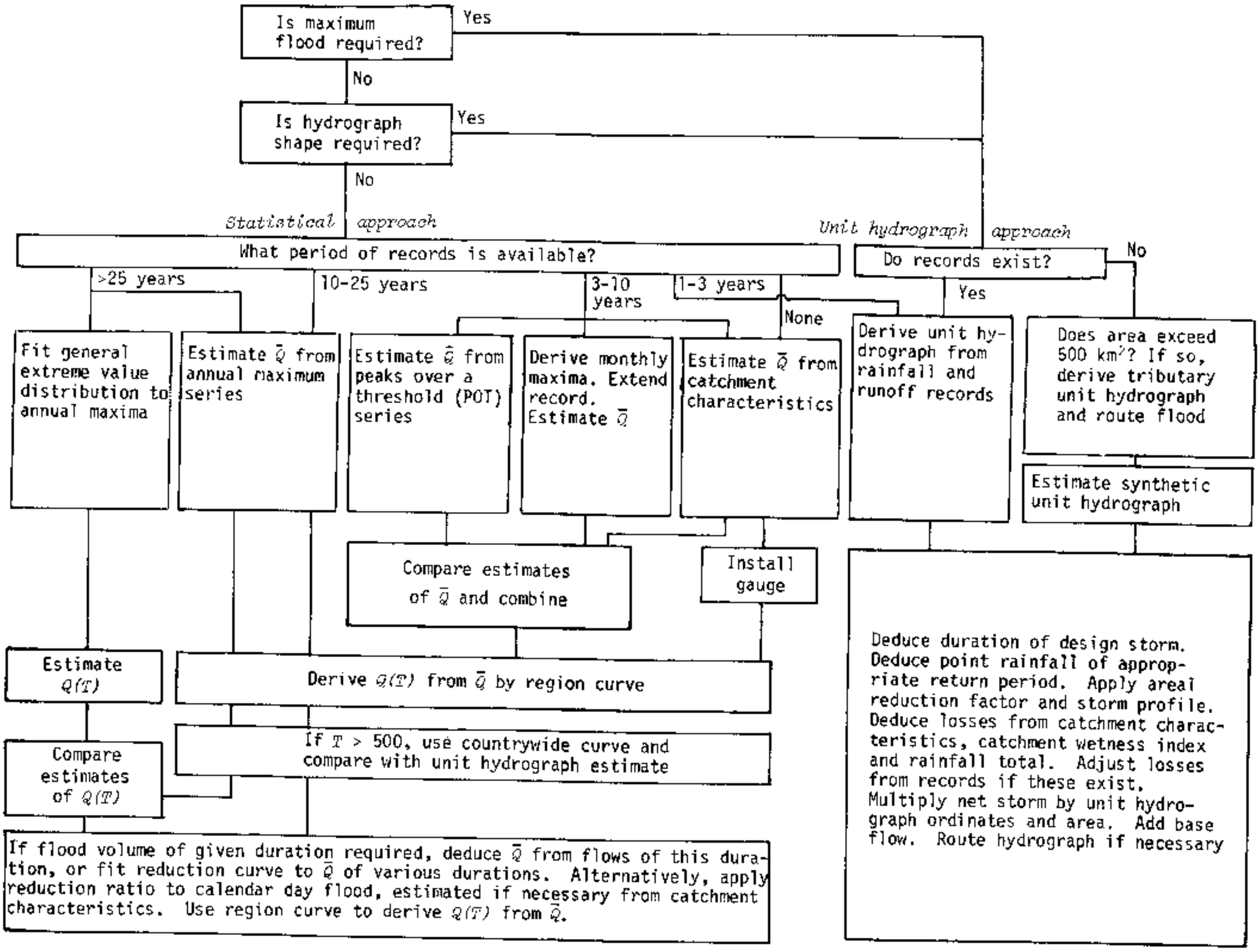

Figure A.1 Estimation of design flood

Where there is a choice between the statistical approach and the unit hydrograph approach, the choice must be a matter of judgement in the light of circumstances. It is possible to put confidence limits on estimates of the mean annual flood, derived from analysis of records or from catchment characteristics, but the formula for the standard error of estimate of the T year flood requires certain assumptions. On the other hand, the estimation by unit hydrograph methods includes errors of estimate of areal rainfall, losses, and unit hydrograph shape, and it has not been possible to quantify the combined errors. However, errors in flood frequency curves simulated by unit hydrograph techniques did not differ appreciably from those obtained by statistical methods when compared with actual records.

In favour of the unit hydrograph approach is that it is based on the analysis of a large number of long-term rainfall stations, and thus uses more records than the statistical analysis of flow records which are relatively short-term. A disadvantage is that it derives a model converting rainfall to runoff largely from moderate storms and floods and applies this model to rare storms or combinations of storm and antecedent condition. On the other hand, the statistical approach applies a distribution or growth curve outside the range of records at any single station. Apart from such theoretical considerations, there are certain practical points which influence the choice of method. The unit hydrograph method can be adjusted to a short period of site records and in general allows the design engineer to see his assumptions and to use judgement based on local records. On the other hand the statistical approach is quicker to apply, can more easily be reduced to a set of rules and does not require rainfall records.

To sum up, the unit hydrograph and statistical approaches are complementary and their errors are comparable. On balance, the unit hydrograph approach may be favoured for use by experienced hydrologists at single sites with relevant rainfall records while the statistical approach may gain if a number of estimates are to be made rapidly at sites with long flow records on large catchments, especially by staff with a simple set of instructions. However, much of the work of annotating maps, deriving catchment characteristics, inspecting gauging stations and collecting records is common to both approaches and the experienced hydrologist is likely to use both approaches and compare the results.

As pointed out earlier, it is often useful to mix the techniques, for instance by plotting the level of the estimated maximum flood on a flood frequency curve. Indeed, as shown later in Section 4, a hybrid approach is used to provide a rapid preliminary method of deriving an estimated maximum flood.

3 STATISTICAL METHODS OF FLOOD ESTIMATION

INTRODUCTION

The statistical approach is the simpler technique to use if it is the flood Q(T) of a finite return period which is required. The purpose of the statistical analysis is to derive a relation between flood magnitude and return period. The return period is the average interval between years with a flood exceeding a given magnitude and is therefore the reciprocal of the probability of a flood exceeding this magnitude occurring in any one year.

The relation between flood peak Q and return period T may be derived from two alternative series of flood peaks. The partial duration series or series of peaks over a threshold is the series of all flood peaks exceeding an arbitrary threshold, q_o. It is not universally used because of the difficulty of defining independence between peaks which occur close together. The annual maximum series consists of the highest flood peaks in each year. Although the return period T deduced from the annual maximum series differs from that deduced from the partial duration series, because some of the higher floods may not be the highest in the year, the difference in T is only about 0.5 and the two series agree closely for large return periods.

Statistical analysis gives the relation between Q and probability of occurrence expressed by a distribution function $F(q) = PR\ (Q \leq q)$. The probability of exceedance is $1 - F(q)$ and the return period $T = 1/(1-F(q))$. The form of $F(q)$ depends on the kind of series and cannot be deduced from theoretical reasoning without empirical studies. The exponential distribution is recommended for the partial duration series and the general extreme value distribution is recommended for the annual maximum series.

An individual annual maximum series may be ranked from largest to smallest and then paired against plotting positions, y, which are related to both frequency and the return period and may be read from appropriate tables, eg Table I.1.16; Q is plotted against y on ordinary graph paper and a return period scale is marked alongside the y axis. Alternatively, probability paper, with the axis graduated in terms of probability, may be used with the plotting position specified as probability or F values, e.g. $F_i = (i-0.44)/(N+0.12)$ for Gumbel paper, where i = 1, 2, ... refers to the smallest, second smallest, ... value in N years of record. A curve or line is drawn to give flows of any return period; a disadvantage is that different curves would be drawn by different analysts. In numerical estimation a form of distribution function is chosen from experience and its parameters are derived from observed data by analytical rules.

If there were several successive periods of record from the same site an estimate of Q(T) could be obtained from each; the standard deviation of these estimates is called the standard error (se) of Q(T) and about two thirds of the estimates fall in the range $Q(T) \pm se(Q(T))$. This standard error is necessarily large for short records, and to overcome this problem the region curve method examines annual maximum series at several stations jointly. The series are made dimensionless by dividing each by its mean $\bar{Q}$, and the relation between $Q(T)/\bar{Q}$ and T is then estimated from the mean pattern of the individual probability plots, with these extended by using the highest floods from groups of geographically well spaced stations which were assumed statistically independent. The region curve takes the form of a general extreme value distribution with different parameters for each region. To apply these curves, the mean annual flood is estimated from a record of flows or in their absence from an equation using catchment characteristics; Q(T) is then obtained by multiplying $\bar{Q}$ by $Q(T)/\bar{Q}$ corresponding to T in a given region.

Both the choice of statistical distribution and the estimation of the constants for parameters of that distribution can be based on the flood record at the design site, always provided this is long enough. In practice, however, it is better that the choice of distribution be based on the study of a number of long term stations. The distributions which have been used in hydrology are described in Chapter I.1 of the Report and it is shown in Chapter I.2 that there is little to choose between the three parameter distributions on goodness of fit tests. Because of its consistency in these tests and because there is some theoretical basis for the choice, the general extreme value distribution is recommended for use. Region curves based on this distribution have been derived from all the annual maximum records in each region; these may be used to derive Q(T) from the mean annual flood $\bar{Q}$ for a site. Where no records exist a preliminary feasibility estimate of $\bar{Q}$ - and thus Q(T) - may be made from catchment characteristics. However, this can only be a very rough estimate and some sort of flow measurement gauge should be installed as soon as a project is proposed. Where there are some records at the site the estimate improves greatly and as the length of record increases so the precision of the estimate improves steadily. Methods of deriving the precision of estimates are given in the Report (I.2.11.5).

Most of the recommended methods of statistical estimation may be applied by following the examples below, though some background study is naturally desirable. Indeed, the better the hydrologist understands the techniques, the easier it will be to improve the estimation by individual judgement. A working knowledge of flood statistics may be obtained from Chapter I.1 of the *Flood Studies Report*.

MEAN ANNUAL FLOOD FROM CATCHMENT CHARACTERISTICS (I.4.3.10)

Where no flow records are available at the site, a preliminary estimate may be made from catchment characteristics. Full details of the characteristics used and their estimation is given in Appendix A to avoid interrupting the argument. The catchment characteristics and notations used are:

Area	AREA	km^2
Stream length	MSL	km
Stream slope	S1085	m/km
Stream frequency	STMFRQ	junctions/km^2
Soil index	SOIL	
Lake index	LAKE	
Urban development	URBAN	
Annual average rainfall	SAAR	mm
Net 1-day rainfall of 5-year return period	RSMD	mm

The estimation of mean annual flood from catchment characteristics is simple once the characteristics have been derived from maps. There are problems with region boundaries and the use of local records, where judgement may be required. Region boundaries are necessarily arbitrary and give rise to apparent anomalies; however, the lower estimates of mean annual flood tend to correspond with steeper region curves. There would be a case for adjustment if adjacent homogeneous basins are consistently under- or over-predicted.

The average country wide equation is

$$\bar{Q} = 0.0201\ \mathrm{AREA}^{0.94}\ \mathrm{STMFRQ}^{0.27}\ \mathrm{S1085}^{0.16}\ \mathrm{SOIL}^{1.23}\ \mathrm{RSMD}^{1.03}\ (1 + \mathrm{LAKE})^{-0.85}$$

but the estimate is improved if regional multipliers are used instead of 0.0201 as follows:

Region	Region no.	Hydrometric areas	Regional multiplier to replace 0.0201 above
Northern Scotland	1	1–16, 88 97, 104 108	0.0186
East Anglia	5	29–35	0.0153
South Coast	7	40 44, 101	0.0234
South west England	8	45–53	0.0315
Central region	2, 3, 4, 9, 10	17 28, 54–87, 102	0.0213
Ireland			0.0172

For the Thames, Lee and Essex areas (Region 6, hydrometric areas 36-39) the appropriate equation is

$$\bar{Q} = 0.373\ \mathrm{AREA}^{0.70}\ \mathrm{STMFRQ}^{0.52}\ (1 + \mathrm{URBAN})^{2.5}. \ *$$

**Note that this 0.373 is incorrectly printed as 0.302 in the FSR.*

Example

As an example of the use of these equations, consider the estimation of the mean annual flood on the Almond at Craigie Hall. This catchment is in the central region so the equation for the mean annual flood is

$$\bar{Q} = 0.0213\ \text{AREA}^{0.94}\ \text{STMFRQ}^{0.27}\ \text{S1085}^{0.16}\ \text{SOIL}^{1.23}\ \text{RSMD}^{1.03}\ (1+\text{LAKE})^{-0.85}$$

The catchment characteristics are derived in Appendix A:

	Value	Log				
AREA	369	2.5670	x	0.94	=	2.4130
STMFRQ	1.02	0.0086	x	0.27	=	0.0023
SOIL	0.459	-0.3382	x	1.23	=	-0.4160
RSMD	32.0	1.5051	x	1.03	=	1.5503
LAKE+1	1.04	0.0170	x	-0.85	=	-0.0145
S1085	4.87	0.6875	x	0.16	=	0.1100
REGION CONST	0.0213	-1.6716	x	1.00	=	-1.6716
			TOTAL			1.9735
		Antilog total				94.1

Mean Annual Flood $\bar{Q}$ = 94.1 cumecs

Remember, however, that when a design flood is required for what is initially an ungauged site, a gauging station and raingauges should be installed at the earliest opportunity. This will then allow a direct estimate to be made by unit hydrograph or by statistical methods before a final figure is required.

MEAN ANNUAL FLOOD BY EXTENSION OF SHORT RECORD (I.3.2)

Given a short record (say 3-10 years) of flows at a site, this may be adjusted for short term fluctuations of estimates of the mean annual flood as follows. Extract the monthly maximum instantaneous flows at the short term station and at nearby long term stations; daily rainfall totals and daily flows or even levels may be included in the long term set. Normalise the data where necessary by taking logarithms, and derive a regression equation for the short term station using the useful long term stations. The choice of regression equation should be made on statistical criteria, supplemented by the use of correlation maps. It will probably be necessary to derive several regression equations for different parts of the extension period to use all the information available.

These regression equations are used to extend the short term monthly maxima backwards until the number of closely related long term stations has fallen to make the prediction equation unreliable, as shown by multiple correlation coefficients. From each complete year of extended record, the highest of the monthly maxima provides an estimate of the annual maximum and the complete series of annual maxima is used to derive the mean annual flood $\bar{Q}$. (Note that this extended series provides a biassed estimate of the variability of annual maxima.) It is not possible to reduce this particular process to rule of thumb, and it is best undertaken by experienced hydrologists. The following example illustrates the technique and the decisions taken in one particular case.

<u>Example</u> Sea Cut at Scarborough (27/33)

At the time of the Floods Study, only 4 years of complete record were available (1965/66-1968/69). Table I.3.1 (pI.265) shows a summary of the bar chart obtained from all records of monthly maximum rainfall and discharge within a 100 km radius. Figure I.3.9 (p I.267) shows the plotted correlation contours. Useful variables appear to be:

- 25/5 complete record back to 1961/62 and incomplete before that to 1958/59;
- 26/2 complete record back to 1961/62;
- 26/801 good correlation but only 7 extra months of record;
- 27/14 correlation a little low but geographically adjacent;
- 27/21 record back to 1959/60;
- 29/1 record back to 1960/61;
- 24724 rainfall correlation very poor;
- 27/802 daily read record almost complete to 1952/53 and good correlation.

It was decided to break the analysis into four periods. As the records of all stations extended over Scarborough's period of flow record (but for a few months) the common period, 1965 to 1969, was used as the basis for all regressions. The stations used to predict each period are shown:

1	1961/62-1964/65	25/5, 26/2, 27/14 27/21, 29/1, 24724, 27/802;
2	1960/61	27/14, 27/21, 29/1, 24724, 27/802;
3	1959/60	27/14, 27/21, 24724, 27/802;
4	1952/53-1958/59	24724, 27/802.

Table I.3.6 (p I.278) shows the regression equations and other relevant details for each prediction equation.

<u>Table I.3.6</u> Scarborough record extension regressions

Possible extension	Coefficients 1 1961/62	2 1960/61	3 1959/60	4 1952/53	Correlation with 27/33
25/5	0.005			—	0.62
26/2	0.698				0.64
27/14	0.492	0.330	0.315	—	0.61
27/21	0.367	0.239	0.609	—	0.76
29/1	0.119	0.462			0.69
24724	0.048	0.292	0.300	0.408	0.08
27/802	0.613	0.638	0.619	0.795	0.79
Intercept	0.322	0.615	0.097	0.352	
R	0.887	0.871	0.845	0.828	—
R^2	0.787	0.759	0.714	0.686	
see %	79, 44	83, 45	91, 48	101, 51	—

The prediction error increased steadily as the number of stations available for extension decreased. It was not thought advisable to use relations 3 and 4 based on only four and two stations, as in both cases most of the correlation was due to the record of 27/802 alone (this is a daily read staff gauge on the Yorkshire Esk).

The regression equations were applied in the period 1965/66-1968/69 to produce a predicted record of annual maxima for comparison with the flows measured during the recorded period. Both equations overestimated in this 4 year period but the effect of including the predicted 1960/61-1964/65 data was to reduce the mean from 52.4 cumecs to 35.3 cumecs.

Table I.3.7 Comparison of recorded and predicted annual maxima

Year	Equation 1	Equation 2	Recorded
1965/66	33.18	35.00	42.46
1966/67	61.66	55.98	59.43
1967/68	63.11	58.75	50.93
1968/69	87.18	117.06	56.62
Mean	61.28	66.70	52.36

Table I.3.8 (p I.279) shows the complete list of Sea Cut's annual maxima. The records of two neighbouring stations are also included for comparison and show that the lower values estimated for the earlier years are not unreasonable.

Table I.3.8 Annual maximum discharges

Water year	Sea Cut 27/33	Bransdale 27/10	Leven Bridge 25/5
1960/61	39.9†	10.3	—
1961/62	14.9†	14.3	22.2
1962/63	30.2†	7.4	52.2
1963/64	13.0†	9.2	30.1
1964/65	10.7†	6.1	26.9
1965/66	42.5	8.7	36.1
1966/67	59.4	5.5	39.0
1967/68	50.9	17.3	68.4
1968/69	56.6	16.8	98.9

†Estimated values.

MEAN ANNUAL FLOOD FROM POT SERIES (I.2.7)

Where a limited period of records is available, the series of peaks over a threshold (POT series) may be used to estimate the mean annual flood using a larger number of flood peaks. This method, like correlation with nearby stations, is particularly appropriate where 3-10 years of record are available.

The data for this method may be obtained as follows. Choose a threshold flow q_o such that on average about three to five peaks a year exceed this threshold. Extract all independent instantaneous peak flows above the threshold, using as a rule for independence that peaks should be separated in time by three times the time to peak and that the flow should decrease between peaks to two thirds of the first peak. List the magnitudes q_i of the M exceedances in N years of record.

A simple statistical model treats the number of exceedances per year as a Poisson variate whose parameter λ is given by

$$\hat{\lambda} = M/N$$

and their magnitudes are treated as an exponential distribution whose parameter β is estimated by

$$\hat{\beta} = \bar{q} - q_o = \sum_{i=1}^{M} (q_i - q_o)/M.$$

Then the T year flow, Q(T) may be estimated from

$$Q(T) = q_o + \hat{\beta} \ln \hat{\lambda} + \hat{\beta} \ln T$$

where ln is the natural logarithm, and specifically $\bar{Q}$ may be estimated from

$$\bar{Q} = q_o + \hat{\beta} \ln \hat{\lambda} + 0.5772\hat{\beta}.$$

Chapter I.2.7 shows that these models are equivalent to the double exponential or Gumbel distribution of the annual maxima. This distribution may not always be appropriate but the method may be used in all cases for estimating the mean annual flood. Its use in practice is simple and is illustrated by the following example.

Example

Assume that an estimate of the mean annual flood of the Almond at Craigie Hall was required in 1961, and that the period of records from 31.8.1956 - 30.9.1960 is available; the POT series which was extracted over 56.50 cumecs gives peak flows as follows:

1955 (part)	88.67
1956	89.09
	95.38
1957	102.10
	121.86
	59.64
	98.39
	162.41
	77.68
	57.43
	108.13
	60.84
1958	65.40
	62.93
1959	63.28
	68.11
	86.63

Average peak = 1467.97 ÷ 17 = 86.35 cumecs

$\hat{\lambda}$ = M/N = number of exceedances ÷ number of whole years = $\frac{2+9+2+3}{4}$ = 4.0

$\hat{\beta} = \bar{q} - q_o$ = average peak - threshold = 86.35 - 56.50 = 29.85 (including part year)

Only the data from whole years are used to estimate $\hat{\lambda}$ but all the floods may be used to estimate $\hat{\beta}$.

$$\begin{aligned}\bar{Q} &= q_o + \hat{\beta} \ln \hat{\lambda} + 0.5772\hat{\beta} \\ &= 56.50 + 29.85\ (\ln 4.0 + 0.5772) \\ &= 56.50 + 29.85\ (1.3863 + 0.5772) \\ &= 56.50 + 58.61 \\ &= 115.11 \text{ cumecs}\end{aligned}$$

MEAN ANNUAL FLOOD FROM ANNUAL MAXIMUM SERIES (I.2.3)

Where an adequate period of records exists, say over 10 years, the mean annual flood may be estimated directly from the annual maxima. The maximum instantaneous flows in each year of record provide the annual maximum series and its arithmetic mean may be used as an estimate of the mean annual flood.

Where this series includes an outlier Q_{max} such that $Q_{max} > 3Q_{med}$, then the mean annual flood should be estimated (see I.2.3.5) from the median annual maximum Q_{med} using $\bar{Q} = 1.07Q_{med}$

Example

Assume that an estimate of the mean annual flood of the Almond at Craigie Hall was required in October 1970. The annual maximum peak flows are listed in the Report, Volume IV, p. 201.

Annual maximum series
1.10.1956-30.9.1970

1956	95.38	1963	123.78
1957	162.41	1964	138.64
1958	65.40	1965	150.31
1959	86.63	1966	93.26
1960	119.71	1967	131.61
1961	162.41	1968	102.76
1962	119.00	1969	177.68
		Sum:	1728.98

Average = 1728.98 ÷ 14

= 123.50

$\bar{Q}$ = 123.50 cumecs

MEAN ANNUAL FLOOD OF VARIOUS DURATIONS (I.5)

The mean annual flood of a given duration, or the flood volume, can be estimated, like the mean annual peak flood, by methods which vary with the available records. If no records exist at the design site, the mean annual calendar day flood, CALMAF, may be estimated from catchment characteristics, and the flood of the desired duration deduced. Where records exist, the maxima for a given duration should be extracted. If short records exist, these may be extended by correlation, or the mean can be estimated by analysis of a POT series of the required duration. The mean may be estimated from a longer annual maximum series. Thus the methods of analysis correspond with those described for peak flows, and do not require further description except for the case of the ungauged site.

Where no records exist at a site the mean annual calendar day flood may be estimated in cumecs from

$$\text{CALMAF} = m\ \text{AREA}^{0.9475}\ \text{STMFRQ}^{0.4068}\ \text{RSMD}^{0.2680}\ \text{SOIL}^{0.7102}$$

with regional multipliers m as follows:

Region	Value
1	0.0395
2	0.0417
3	0.0410
4	0.0360
5	0.0279
6	0.0250
7	0.0428
8	0.0585
9	0.0539
10	0.0459

The mean annual flood of longer duration may be estimated from CALMAF by reference to the two ratios of 3 day and 10 day flood to daily flood (AR3 and AR10), each defined as a flow over the given duration, and to families of reduction curves; AR3 and AR10 are given by

$$\log(\mathrm{AR3}) = -0.101 - 0.081 \log(\mathrm{S1085})$$
$$\log(\mathrm{AR10}) = -0.269 - 0.127 \log(\mathrm{S1085})$$

Example

Estimate the mean annual flood of 2 day and 6 day duration for the Almond at Craigie Hall. For Region 2 the equation for the mean annual calendar day flood is

$$\mathrm{CALMAF} = 0.0417\ \mathrm{AREA}^{0.9475}\ \mathrm{STMFRQ}^{0.4068}\ \mathrm{RSMD}^{0.6280}\ \mathrm{SOIL}^{0.7102}$$

	Value	Log	
AREA	369	2.5670 x 0.9475 =	2.4322
STMFRQ	1.02	0.0086 x 0.4068 =	0.0035
RSMD	32.0	1.5051 x 0.6280 =	0.9452
SOIL	0.459	- 0.3382 x 0.7102 =	- 0.2402
Region multiplier	0.0417	-1.3799 =	- 1.3799
			1.7608

antilog = 57.65

Mean annual calendar day flood = 57.7 cumecs

Using the prediction equations for the ratio of the 3 day and 10 day floods (AR3 and AR10) to the calendar day flood

$$\begin{aligned}\log(\mathrm{AR3}) &= -0.101 - 0.081 \log(\mathrm{S1085})\\ &= -0.101 - 0.081 \log(4.87)\\ &= -0.101 - 0.081\ (0.6875)\\ &= -0.157\end{aligned}$$

$$\begin{aligned}\log(\mathrm{AR10}) &= -0.269 - 0.127 \log(4.87)\\ &= -0.269 - 0.127\ (0.6875)\\ &= -0.356\end{aligned}$$

$$\begin{aligned}\mathrm{AR3} &= 0.697\\ \mathrm{AR10} &= 0.441\end{aligned}$$

These two points are plotted and a reduction curve fitted as in Figure I.5.11.A.

From this the required ratios are estimated to be

AR2 = 0.81, AR6 = 0.54

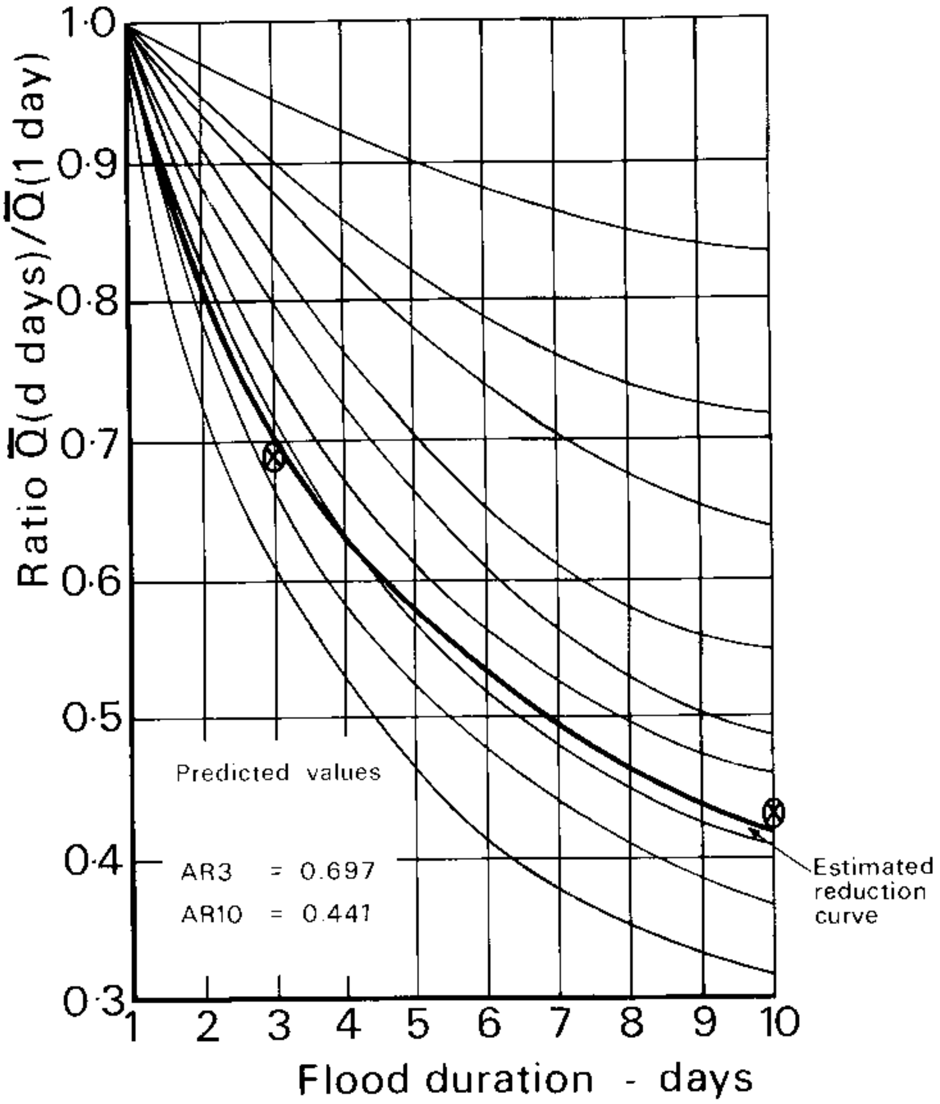

Figure 5.11A Example of use of prediction equations to sketch reduction curve.

Thus the mean annual floods of 2 and 6 days duration are

$$\bar{Q}, \text{ 2 days} = 57.7 \times 0.81 = 46.7 \text{ cumecs}$$

$$\bar{Q}, \text{ 6 days} = 57.7 \times 0.54 = 31.2 \text{ cumecs}$$

Q(T) FROM MEAN ANNUAL FLOOD (I.2.6)

The relationship between Q(T), the flood of return period T, and the mean annual flood $\bar{Q}$, can be derived by statistical analysis of a long record. In the usual case where there is not a record at the design site of more than 25 years, it is suggested that the flood frequency curve should be deduced by using a region curve linking $Q(T)/\bar{Q}$ to T, and scaling up this curve to $\bar{Q}$ as derived in previous sections.

The region curves (Table I.2.39, p 173) were derived from all the flood records in a group of hydrometric areas and are summarised in the table below. For values of T over 500 years it is recommended that a curve based on countrywide records should be used.

These curves were derived from instantaneous peaks, but may also be applied to floods of longer duration where they may be conservative.

Table I.2.39 - Region curve ordinates

		Return period:						
Region	Hydrometric areas	2	5	10	25	50	100	200
1	1-16,88-97,104-108	0.90	1.20	1.45	1.81	2.12	2.48	2.89
2	17-21,77-87	0.91	1.11	1.42	1.81	2.17	2.63	3.18
3	22-27	0.94	1.25	1.45	1.70	1.90	2.08	2.27
4	28,54	0.89	1.23	1.49	1.87	2.20	2.57	2.98
5	29-35	0.89	1.29	1.65	2.25	2.83	3.56	4.46
6/7	36-44,101	0.88	1.28	1.62	2.14	2.62	3.19	3.86
8	45-53	0.88	1.23	1.49	1.84	2.12	2.42	2.74
9	55-67,102	0.93	1.21	1.42	1.71	1.94	2.18	2.45
10	68-76	0.93	1.19	1.38	1.64	1.85	2.08	2.32
Great Britain		0.89	1.22	1.48	1.88	2.22	2.61	3.06
Ireland		0.95	1.20	1.37	1.60	1.77	1.96	2.14

Example

Estimate the 50 year peak flood for the Almond, using the estimate of $\bar{Q}$ based on catchment characteristics

$$Q(50) = \bar{Q} \times 2.17 = 94.1 \times 2.17 = 204 \text{ cumecs}$$

Q(T) FROM ANNUAL MAXIMUM SERIES (I.2.11)

Where ample records exist, it is possible to estimate Q(T) directly from annual maximum flows. Given the set of annual maxima comprising the highest instantaneous flows in each year of record, it is suggested that a general extreme value distribution may be fitted to this set. The three parameters u, α and k of this distribution may be estimated graphically or by moments, sextiles or maximum likelihood as described in Section I.1.3.4. However, it will be extremely rare for records to be adequate to provide reasonable estimates of three parameters. It is therefore suggested that, for records between 10 and 25 years, an extreme value Type 1 distribution may be used for return periods up to 2N, where N is the number of years of record. For records over 25 years, a general extreme value distribution may be fitted for return periods up to 2N. In both cases region or countrywide curves should be used for higher return periods.

An alternative is to fit a general extreme value distribution with shape factor k corresponding to the region curve and to estimate the other two parameters from the records by moments.

Details of these estimation procedures cannot be demonstrated by numerical examples. However, programs for the analysis are available.

4 UNIT HYDROGRAPH METHOD OF FLOOD ESTIMATION

INTRODUCTION

Unit hydrograph methods are appropriate where the shape and volume of a flood hydrograph are needed, as for routing through a reservoir, or where an estimate of the maximum flood is required. In general, the procedure is rather more complex than the statistical methods outlined previously but the number of alternatives is less.

The unit hydrograph for the design site should be derived if possible from rainfall and runoff records but may be estimated from catchment characteristics if no records exist. However, as with the statistical approach, estimates are much improved if actual records can be used. Estimates based on catchment characteristics alone should be regarded as approximate.

The duration of the design storm is given by the unit hydrograph dimensions and the point rainfall is estimated for a return period which depends on the return period of the design flood. An areal reduction factor is applied to give the catchment rainfall total, and a time profile is provided.

The proportion of the design storm providing immediate runoff is calculated from an equation involving soils, an appropriate antecedent condition and the rainfall total. This estimate can be adjusted to take account of records at the site. The net storm is multiplied by the ordinates of the unit hydrograph to give the design flood, with base flow added.

This technique may be supplemented by a simulation procedure which gives estimates based on sampling the various conditions rather than using a single choice. Because unit hydrographs may be derived from a short period of record, these techniques provide a comparison with the statistical approach for short records. For return periods over 500 years, where region curves become increasingly ill-defined, unit hydrograph techniques should be used to supplement statistical estimates based on the country-wide curve.

An estimate of the maximum flood follows similar lines except that more conservative assumptions should be made and an allowance is required for snowmelt.

Before the detailed procedure is given with a numerical example, a brief summary may be useful.

The time to peak in hours of the unit hydrograph may be estimated from

$$T_p = 46.6(\text{MSL})^{0.14}(\text{S1085})^{-0.38}(1 + \text{URBAN})^{-1.99}(\text{RSMD})^{-0.4}$$

where there are no records. Where some records exist, T_p may be estimated more reliably from the lag of the catchment, defined as the time from the centroid of rainfall to the peak runoff or centroid of peaks, using

$$T_p = 0.9\text{LAG}$$

or by deriving a unit hydrograph from records of rainfall and runoff as described later. The peak of the unit hydrograph Q_p in cumecs/100 km² is estimated from

$$Q_p = 220/T_p$$

and its time base, TB, as $2.52T_p$.

Thus a triangular unit hydrograph can be drawn up from these three parameters, T_p, Q_p and TB.

The duration D of the design storm depends on T_p and the mean annual rainfall SAAR by

$$D = (1 + \text{SAAR}/1000)T_p.$$

The return period of the design storm is deduced from the return period of the design flood using Figure I.6.61. This relationship between design storm and flood return period is the result of a statistical sampling exercise and allows the design flood of any required return period to be computed. It is of course not suggested that <u>all</u> storms with for instance an 81-year return period will necessarily produce the 50 year return period flood peak but that the given combination of design storm depth, duration, profile and antecedent conditions specified here will give the best estimate. The mean point rainfall of duration D and return period 5 years is derived for a given catchment from the mean point rainfall of 2 day duration and 5 year return period (2 day M5) given by Figure II.3.2, and the ratio M5(D)/2 day M5 (Table II.3.7). Then the growth factor MT/M5 (Table II.2.7 or II.2.9) is used to estimate the mean point rainfall P. This is converted to a catchment rainfall by using an areal reduction factor (Figure II.5.1), and the catchment storm of duration D is distributed in time by the standard profile (Figure I.6.65).

The appropriate design catchment wetness index CWI is estimated from Figure I.6.62 and the percentage runoff is estimated from

$$PR = 95.5SOIL + 12URBAN + 0.22(CWI-125) + 0.1(P-10).$$

This percentage runoff may be applied to the design storm and the resulting ordinates are multiplied by the unit hydrograph, adjusted for catchment area. The non-separated flow or baseflow in cumecs/km^2 may be added from

$$ANSF/AREA = 0.00033(CWI-125) + 0.00074RSMD + 0.003.$$

Where records exist not only can the unit hydrograph be deduced but also the PR may be modified from comparisons of rainfall and response runoff.

When maximum flood estimates are required, this approach is modified through the rainfall total and profile, while T_p should be reduced and thus Q_p increased, and a CWI of 125 is selected for the start of the storm. An allowance for snowmelt runoff is added.

UNIT HYDROGRAPH DERIVATION

If adequate rainfall and river flow records exist at the design site or somewhere near it on the same river, it is recommended that a number of unit hydrographs be determined. A minimum sample of five events is suggested but it is more important to use only those events where the rainfall distribution is either spatially uniform or at least typical of the topography. It can sometimes be misleading to concentrate on clean looking isolated hydrographs; they may arise from small quantities of runoff perhaps originating uncharacteristically from only a part of the catchment.

Unit hydrographs should be derived from, say, the five largest events with spatially near-uniform rainfall. Their peak values and times to peak should be averaged. The peaks should be aligned and average values calculated for the ordinates on either side. The several unit hydrographs will probably differ markedly but any search for a systematic variation with some measure of storm intensity should be done with caution (I.6.5.3).

APPLICATION OF UNIT HYDROGRAPH METHODS

Chapter I.6 shows how a design hydrograph, with a peak of specific return period, is provided for an ungauged catchment. The data used were gathered from catchments under 500 km^2 in the United Kingdom and were mostly from events less than twice the mean annual flood. Application to larger catchments, to overseas catchments, or to the prediction of floods with higher return periods, must be made with caution. The area limit was a product of requirements for analysis but in design it is quite normal to consider spatially uniform rainfall over much larger areas than 500 km^2. Indeed, any consideration of different rain amounts falling on subcatchments raises the question of assessing the probability of various combinations of such different amounts. It is fairly safe to use the methods described for catchments up to 1000 km^2 but for larger catchments it would be advisable to subdivide the catchment. Wherever possible try to derive the unit hydrograph from events with reasonably uniform rainfall. Where this is not possible, or where the records do not exist and a synthetic unit hydrograph is required, it will be necessary to estimate design floods at sites on tributaries and to route these down to the design site by appropriate flood routing methods.

For a preliminary answer to the estimated maximum flood and for instance in selecting sites for detailed study, a rapid method of prediction is available relating the estimated maximum flood to area, net short-term rainfall, soil index, slope and urban fraction. This formula[1], the result of research since the Flood Studies Report was published, was derived by estimating the maximum flood for 80 catchments using an observed unit hydrograph and standard percentage runoff, and then relating these estimates to catchment characteristics. The formula lacks the precision that can be obtained by rapid application of the unit hydrograph method, even where no local records can be used, but has the merit of simplicity.

Design floods may be estimated for sites on tributaries and routed down to the design site. The choice of flood routing method depends on an estimate of the attenuation of the peak flow (Volume III, Section 5.1) and this estimate, together with the design requirements, makes it possible to decide (Figure III.4.1) whether the Muskingum-Cunge method (III.5.2) is adequate or whether the variable parameter diffusion method (III.5.3) is necessary.

As regards overseas catchments, the main difficulty is in extracting catchment characteristics to the same specification. However, if soil index and stream slope can be assessed with reasonable confidence and the rainfall statistics are available, there is every reason to suppose that the method can be applied to catchments in other cool temperate zones.

Specification of peak flows with return periods much greater than 1000 years can only be done with a large error of estimate. This applies to statistical methods as well as the semi-deterministic procedure described here. The difficulties in analysing very large floods, which are almost never measured properly in respect of either rainfall or runoff, necessitate the adoption of conservative assumptions when designing against maximum floods.

For nearly all design purposes engineers prefer to specify the return period of a flood peak or volume. In the design of dam spillways where underestimation could involve loss of life, the maximum flood may have to be considered. This extreme event may also be examined, together with floods of specific return periods, to build up a complete picture of the flood regime.

A DESIGN HYDROGRAPH WITH PEAK OF SPECIFIED RETURN PERIOD

The following examples are presented mainly for the situation without rainfall or runoff data from the catchment or nearby catchments. If such data exist, the procedure may be modified to varying degrees and this is illustrated at the relevant points within the examples.

Figure I.6.55 presents, in the form of a flow chart, the several steps in the estimation of a design hydrograph (references are to Volume I of the Report). A more detailed description now follows with an example of the calculations for the River Almond. The specific numerical values are given on the right of the page alongside the relevant description of the general procedure.

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[1] F A K Farquharson, M J Lowing and J V Sutcliffe, Some aspects of design flood estimation, BNCOLD/University of Newcastle upon Tyne Symposium on inspection, operation and improvement of existing dams, 1975.

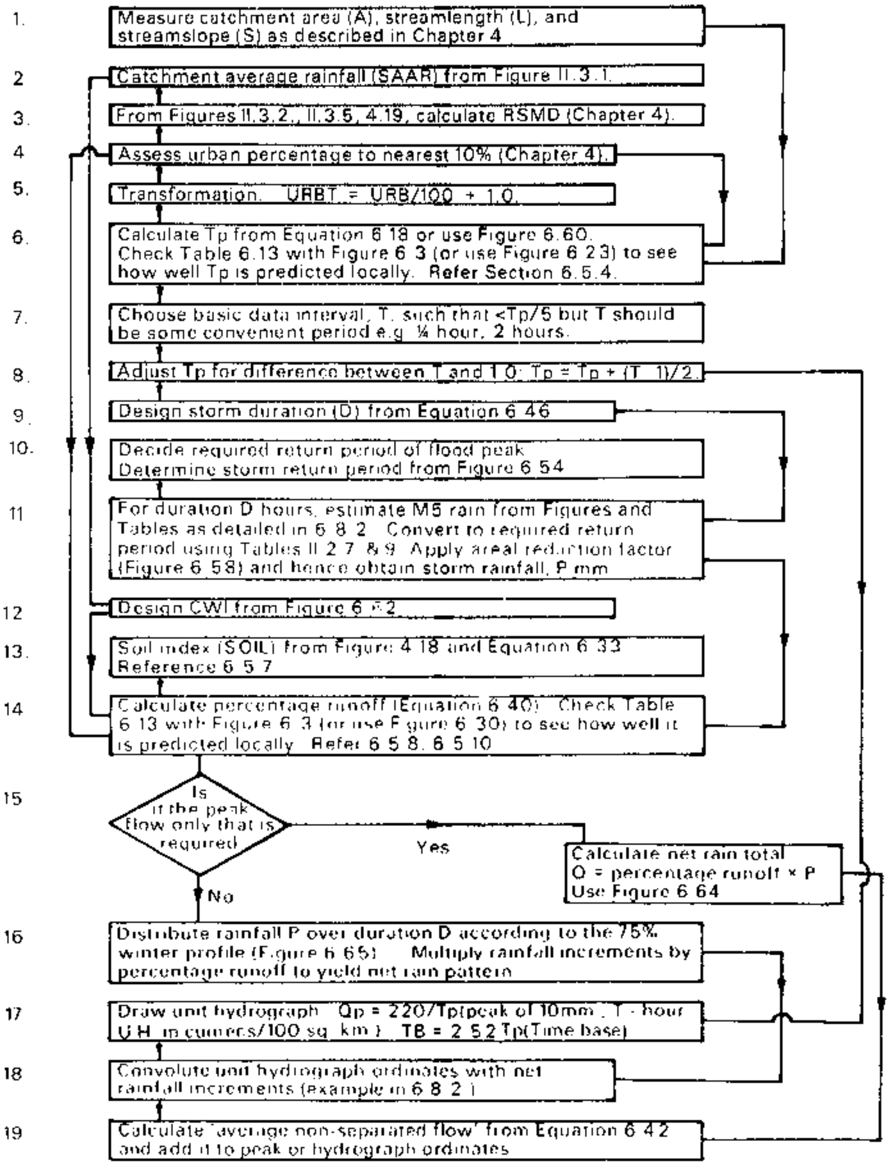


Figure I.6.55 Flow chart of design procedure for flood with peak of specified return period

Step 1

Define topographic catchment on 1:25 000 map and measure area (A) by any convenient method	A = 369 km^2
Length (MSL) of main stream on 1:25 000 map should be measured with dividers set to 0.1 km step length. Other methods of measurement will give different answers and should not be used	MSL = 44.6 km

Channel slope is the average slope (S1085) between points 10 and 85% of the length of the main stream measured up from the outlet, ie S1085 = (H(85%) - H(10%))/0.75MSL where H ( ) are the altitudes (m) as interpolated between the contours

S1085 = 4.87 m/km

Step 2

The average annual rainfall SAAR is obtained from Figure II.3.1. The average for the catchment may be obtained by sampling at about 20 points equally spaced on a grid overlay and taking the arithmetic mean. Alternatively, the weighted areas technique can be used

SAAR = 914 mm

Step 3

The calculation of RSMD requires M5-2 day rainfall from Figure II.3.2, the ratio r (M5-60 min/M5-2 day) from Figure II.3.5 and the effective mean SMD (SMDBAR) from Figure I.4.19. In all three cases catchment average values are required and these may be obtained by grid point sampling or weighted areas.

The relationship between RSMD and SAAR (standard annual average rainfall) shown in Figure I.6.59 allows an initial estimate of RSMD to be made, as does the map reproduced in Appendix A. The ratio M5-24 hours/M5-2 day is determined in terms of r, from Table II.3.7 (reproduced here as Table 6.21). M5-24 hours follows and this is converted to M5-1 day by dividing by 1.11 from Table II.3.1. The 1 day areal reduction factor (ARF) is obtained from Figure II.5.1 reproduced here as Figure 6.58.

r	=	25%
M5-24h/M5-2 day	=	82%
M5-2 day	=	57 mm
M5-24 hours	=	46.7 mm
M5-1 day	=	42 mm
ARF (1 day)	=	0.92
M5-1 day x ARF	=	38.6 mm
SMDBAR	=	6.6 mm
RSMD	=	32.0 mm

RSMD = M5-1 day x ARF - SMDBAR

D (hours) 1	2	4	6	12	24	48
12	18	26	33	49	72	106
15	21	30	37	53	75	106
18	25	34	41	56	77	106
21	28	38	45	60	80	106
24	31	41	48	63	81	106
27	35	44	51	65	83	106
30	38	48	55	68	85	106
33	41	51	57	71	87	106
36	44	54	60	73	88	106
39	47	57	63	75	89	106
42	50	60	66	77	91	106
45	53	63	68	79	92	106

Table 6.21
Ratio M5-D hour/M5-2 day in terms of ratio r (D=1 hour) based on Table II.3.7. Ratios are shown as percentag

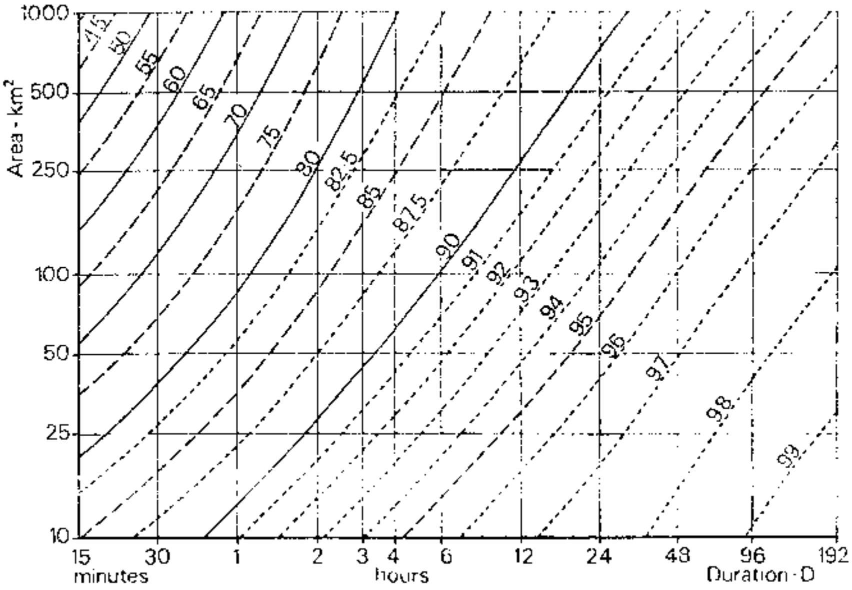


Figure 6.58 Areal reduction factor - part reproduced from Figure II.5.1.

Step 4

The urban fraction of the catchment (URBAN) should be assessed from any convenient map. An estimate of the 'grey' areas on the 1:63 360 map or 'orange' areas on 1: 50 000 map is adequate

URBAN = 0.114

Step 5

The urban percentage variable is transformed for use in Step 6.

URBT = 1.0 + URBAN

URBT = 1.114

Step 6

The time to peak of the 1 hour unit hydrograph is calculated from Equation 6.18

$$T_p = 46.6\,MSL^{0.14}\,S1085^{-0.38}\,URBT^{-1.99}\,RSMD^{-0.4}$$

$$\begin{aligned} T_p &= 46.6 \times 44.6^{0.14} \\ &\quad \times 4.87^{-0.38} \\ &\quad \times 1.114^{-1.99} \\ &\quad \times 32.0^{-0.4} \\ &= 46.6 \times 1.702 \\ &\quad \times 0.548 \times 0.8067 \\ &\quad \times 0.25 \\ &= 8.8 \text{ hours} \end{aligned}$$

If a direct estimate of LAG is available a better estimate of T_p follows from Equation I.6.14.

$T_p = 0.9$ LAG

In this example, the no data estimate of $T_p = 8.8$ hours will be used

If an improved estimate is not available it is advisable to refer to Figure I.6.23 to see if T_p is badly predicted locally.
If so, a correction to predicted T_p might be indicated.
Although catchment shape and overland slope factors did not enter the T_p regression it is, of course, better to ensure that neighbouring catchments are of broadly similar configuration before applying such corrections

Because analysis was based on catchments without lake storage, the presence of lakes or reservoirs may require a modification to T_p (see also I.7.4.2)

The area draining through reservoirs was only 4%

Step 7

The basic data interval, T hours, should be chosen such that

$T \simeq T_p/5$

but T should also be some convenient number of hours or fraction of an hour

T = 1.0 hour

Step 8

If necessary, the value of T_p must be adjusted for application to the T hour unit hydrograph rather than the 1 hour unit hydrograph. This may be done in the same way that unit hydrographs were all standardised to 1 hour in the analyses (Section I.6.4.8).

new T'_p = old $T_p + (T-1)/2$

Step 9

The recommended design storm duration (Section I.6.7.6) is obtained from Equation I.6.46,
$D = (1.0+\text{SAAR}/1000)T_p$
but it is convenient to take the nearest odd integer multiple of T

$D = (1 + \frac{914}{1000})\ 8.8$
$= 16.8$ hours

D = 17 hours

Note: Use T_p' if $T \neq 1$

In the case of a spillway design, where it is the reservoir outflow peak which is to be synthesised, it is suggested that the design storm duration should be increased by first adding the reservoir lag to T_p, ie,

$D = (1 + SAAR/1000)\ (T_p + \text{RESERVOIR LAG})$

Step 10

The required return period of the flood must be decided at this stage	say, 50 years
The recommended storm return period (Section I.6.7.6) is obtained from Figure I.6.61 (based on Figure I.6.54)	SPR = 81 years

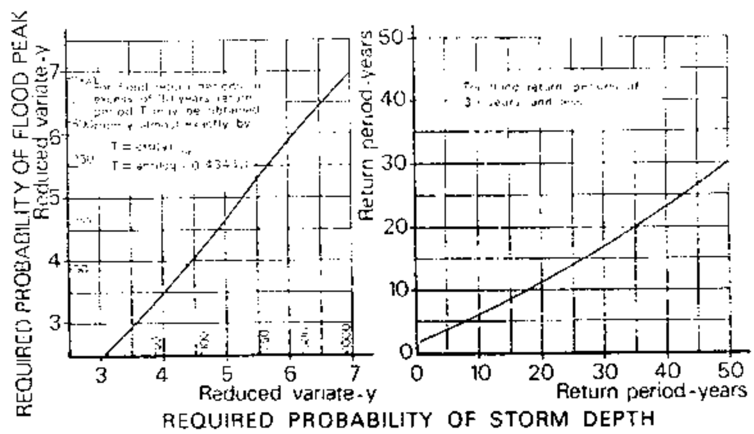


Figure 6.61

Recommended storm return period to yield flood peak of required return period by design method.

Flood peak return period	2.33	5	10	20	30	50	100	250	500	1000
Rainfall return period	2	8	17	35	50	81	140	300	520	1000

Step 11

The particular combination of figures and tables required for calculating the design rainfall depends mainly on the duration but this will usually be less than 48 hours for which the following applies

From Table II.3.7 (reproduced here as Table 6.21) the ratio M5(D)/M5-2 day may be extracted in terms of r (found in Step 3)	M5-17 hours/M5-2 day = 72%
This ratio is applied to M5-2 day, which has also been calculated in Step 3, to give M5(D)	M5 (17 hours) = 41.0 mm

Table II.2.7

Growth factors MT/M5 for England and Wales

M5 (mm)	Partial duration series 2M	1M	M2	Annual maximum series M10	M20	M50	M100	M1000	M10 000
0.5	0.52	0.67	0.76	1.14	1.30	1.51	1.70	2.52	3.76
2	0.49	0.65	0.74	1.16	1.32	1.53	1.74	2.60	3.94
5	0.45	0.62	0.72	1.18	1.35	1.56	1.79	2.75	4.28
10	0.43	0.61	0.70	1.21	1.41	1.65	1.91	3.09	5.01
15	0.46	0.62	0.70	1.23	1.44	1.70	1.99	3.32	5.54
20	0.50	0.64	0.72	1.23	1.45	1.73	2.03	3.43	5.80
25	0.52	0.66	0.73	1.22	1.43	1.72	2.01	3.37	5.67
30	0.54	0.68	0.75	1.21	1.41	1.70	1.97	3.27	5.41
40	0.56	0.70	0.77	1.18	1.37	1.64	1.89	3.03	4.86
50	0.58	0.72	0.79	1.16	1.33	1.58	1.81	2.81	4.36
75	0.63	0.76	0.81	1.13	1.27	1.47	1.64	2.37	3.43
100	0.64	0.78	0.83	1.12	1.24	1.40	1.54	2.12	2.92
150	0.64	0.78	0.84	1.11	1.21	1.33	1.45	1.90	2.50
200	0.64	0.78	0.84	1.10	1.19	1.30	1.40	1.79	2.30
500	0.65	0.79	0.85	1.09	1.15	1.20	1.27	1.52	
1000	0.66	0.80	0.86	1.07	1.12	1.18	1.23	1.42	—

Table II.2.9

Growth factors MT/M5 for Scotland and Northern Ireland

M5 (mm)	Partial duration series 2M	1M	M2	Annual maximum series M10	M20	M50	M100	M1000	M10 000
0.5	0.55	0.68	0.76	1.14	1.30	1.51	1.71	2.54	3.78
2	0.55	0.68	0.76	1.15	1.31	1.54	1.75	2.65	4.01
5	0.54	0.67	0.76	1.16	1.34	1.62	1.86	2.94	4.66
10	0.55	0.68	0.75	1.18	1.38	1.69	1.97	3.25	5.36
15	0.55	0.69	0.75	1.18	1.38	1.70	1.98	3.28	5.44
20	0.56	0.70	0.76	1.18	1.37	1.66	1.93	3.14	5.12
25	0.57	0.71	0.77	1.17	1.36	1.64	1.89	3.03	4.85
30	0.58	0.72	0.78	1.17	1.35	1.61	1.85	2.92	4.60
40	0.59	0.74	0.79	1.16	1.33	1.56	1.77	2.72	4.16
50	0.60	0.75	0.80	1.15	1.30	1.52	1.72	2.57	3.85
75	0.62	0.77	0.82	1.13	1.26	1.45	1.62	2.31	3.30
100	0.63	0.78	0.83	1.12	1.24	1.40	1.54	2.12	2.92
150	0.64	0.79	0.84	1.10	1.20	1.33	1.45	1.90	2.50
200	0.65	0.80	0.85	1.09	1.18	1.30	1.40	1.79	2.30
500	0.66	0.80	0.86	1.08	1.14	1.20	1.27	1.52	—
1000	0.66	0.80	0.86	1.07	1.12	1.18	1.23	1.42	-

From Table II.2.7 & 9, reproduced here, the growth factor MT/M5 is assessed for the storm return period and hence MT is estimated

Growth factor = 1.70

M81 = 70.0 mm

(Table II.2.7 applies to England and Wales, Table II.2.9 to Scotland and N. Ireland)

This is a point rainfall estimate and is reduced to a catchment average estimate by applying an areal reduction factor obtained from Figure I.6.58

ARF = 0.90

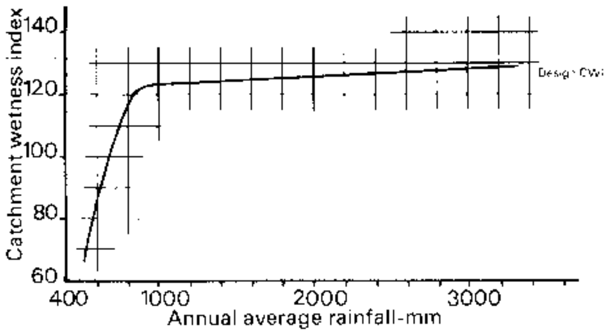
The storm return period rain in D hours over the catchment = ARF x MT = P

P = 63.0 mm

Step 12

The recommended antecedent catchment condition (Section I.6.7.6) is expressed by the design CWI which is read from Figure I.6.62 (based on Figure I.6.44 and Equation I.6.43) CWI = 123

Figure I.6.62 Recommended design values for catchment wetness index



Step 13

The soil index (SOIL) is derived in Appendix A from the fractions of the catchment occupied by the various soil classes S_1 to S_5 SOIL = 0.459

Step 14

The standard percentage runoff (SPR) is calculated from
SPR = 95.5SOIL + 12URBAN

SPR = 95.5(0.459) + 12(0.114)
= 45.2%

The percentage runoff appropriate to the design event is calculated as
PR = SPR + 0.22(CWI-125) + 0.1(P-10)
(Equation I.6.40 from Section I.6.5.8)

PR = 50.1% for P = 63.0 mm

At this point runoff data from similar areas nearby may be used to modify the estimate

Step 15

If the peak flow is all that is required, it may be determined directly from Figure I.6.64. The design duration and T_p are divided by the basic data interval T. With the recommended design storm profile, the peak flow (response runoff only) per unit area per unit (net) rainfall is dependent on D and T_p only.

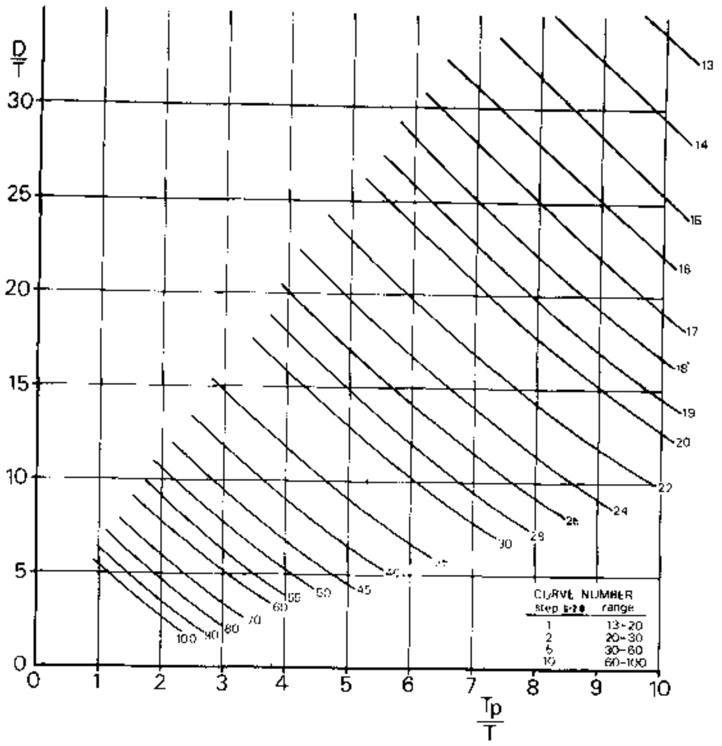


Figure 6.64

Diagram for direct determination of peak of synthesized response flow hydrograph.

The peak flow is derived from the curve number (CN) of Figure I.6.64 thus:

$$\hat{q} = \frac{CN \times AREA \times P \times PR}{10^5 \times T}$$

$D/T = 17$
$T_p/T = 8.8$
$CN = 19.2$
$\hat{q} = 223.6$ cumecs

Steps 16 to 18 are needed to describe the complete hydrograph. Otherwise go to Step 19

Step 16

The rainfall P is distributed over duration D according to the 75% winter profile of Table II.6.3. This is reproduced here, in graphical form, as Figure I.6.65

In the example case, T is 1 hour and D is 17 hours; the increments at 1 hour intervals should therefore be read from Figure 6.65 as differences between successive multiples of 5.9% (100/17) of the duration.

Table 6.23 gives the design storm profile and, after applying the constant percentage runoff PR, the net rain profile

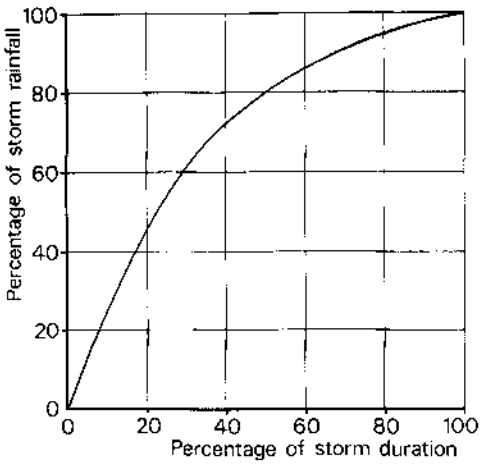


Figure 6.65

Recommended design storm profile - the 75% winter.

Table 6.23 Design storm and net rain distribution for 50 year flood - Almond at Craigie Hall (interval length = 1 hour)

Percentage of Duration	5.9	17.6	29.4	41.2	52.9	64.7	76.4	88.2	100
Rain %	16	42	60	74	82	89	94	98	100
Increment %	16	26	18	14	8	7	5	4	2
Increment in mm	10.1	16.4	11.3	8.8	5.0	4.4	3.2	2.5	1.3

Interval	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Total Rain	0.6	1.3	1.6	2.2	2.5	4.4	5.7	8.2	10.1	8.2	5.7	4.4	2.5	2.2	1.6	1.3	0.6
Net Rain	0.3	0.7	0.8	1.1	1.3	2.2	2.8	4.1	5.1	4.1	2.8	2.2	1.3	1.1	0.8	0.7	0.3

Step 17

The simple triangular unit hydrograph may be drawn

$Q_p = 220/T_p$ cumecs/100 km^2

Time base = $2.52 \times T_p$

$Q_p = 25.0$ cumecs/100 km^2

TB = 22.2 hours

Figure 6.66A shows the synthetic triangular unit hydrograph for the Almond catchment

Alternative unit hydrographs may be substituted at this point if there are some suitable data from the catchment

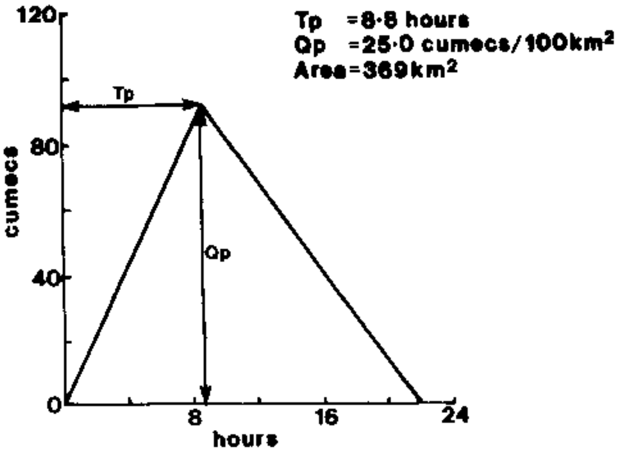


Figure 6.66A

Synthetic triangular unit hydrograph - River Almond

Step 18

The convolution of the unit hydrograph with the net rainfall pattern may be set out as a table

Table 6.24A illustrates the computation for the example catchment

The hourly ordinates of the unit hydrograph are multiplied by AREA/100 to correct for the catchment area and are set out in the first row of the table. The left hand column shows the net rain increments divided by 10. (The unit hydrograph is for 10 mm of rain) The unit hydrograph ordinates are all multiplied by the first net rain increment and the results entered directly below. They are then multiplied by the second increment and the results displaced one column to the right and so on. The column sums give the response runoff hydrograph. As the profile is symmetrical and the unit hydrograph a simple triangle, the peak of the response runoff hydrograph always occurs in the column where the peak unit hydrograph ordinate is multiplied by the biggest net rain increment. Because of this it was possible to provide Figure I.6.64 for direct evaluation of the peak alone

NET RAIN cm	UNIT HYDROGRAPH (Cumecs) 0	10.5	21.0	31.6	42.1	52.6	63.1	73.6	84.2	91.2	84.3	77.4	70.5	63.6	56.7	49.8	42.8	35.9	29.0	22.1	15.2	8.3	1.4
0.03	0	0.3	0.6	0.9	1.3	1.6	1.9	2.2	2.5	2.7	2.5	2.3	2.1	1.9	1.7	1.5	1.3	1.1	0.9	0.7	0.5	0.2	0.0
0.07		0	0.7	1.5	2.2	2.9	3.7	4.4	5.2	5.9	6.4	5.9	5.4	4.9	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.1	0.6
0.06			0	0.8	1.7	2.5	3.4	4.2	5.0	5.9	6.7	7.3	6.7	6.2	5.6	5.1	4.5	4.0	3.4	2.9	2.3	1.8	1.2
0.11				0	1.2	2.3	3.5	4.6	5.8	6.9	8.1	9.3	10.0	9.3	8.5	7.8	7.0	6.2	5.5	4.7	3.9	3.2	2.4
0.13					0	1.4	2.7	4.1	5.5	6.8	8.2	9.6	10.9	11.9	11.0	10.1	9.2	8.3	7.4	6.5	5.6	4.7	3.8
0.22						0	2.3	4.6	7.0	9.3	11.6	13.9	16.2	18.5	20.1	18.5	17.0	15.5	14.0	12.5	11.0	9.4	7.9
0.28							0	2.9	5.9	8.8	11.8	14.7	17.7	20.6	23.6	25.5	23.6	21.7	19.7	17.8	15.9	13.9	12.0
0.41								0	4.3	8.6	13.0	17.3	21.6	25.9	30.2	34.5	37.4	34.6	31.7	28.9	26.1	23.2	20.4
0.51									0	5.4	10.7	16.1	21.5	26.8	32.2	37.5	42.9	46.5	43.0	39.5	36.0	32.4	28.9
0.41										0	4.3	8.6	13.0	17.3	21.6	25.9	30.2	34.5	37.4	34.6	31.7	28.9	26.1
0.28											0	2.9	5.9	8.8	11.8	14.7	17.7	20.6	23.6	25.5	23.6	21.7	19.7
0.22												0	2.3	4.6	7.0	9.3	11.6	13.9	16.2	18.5	20.1	18.5	17.0
0.13													0	1.4	2.7	4.1	5.5	6.8	8.2	9.6	10.9	11.9	11.0
0.11														0	1.2	2.3	3.5	4.6	5.8	6.9	8.1	9.3	10.0
0.08															0	0.8	1.7	2.5	3.4	4.2	5.0	5.9	6.7
0.07																0	0.7	1.5	2.2	2.9	3.7	4.4	5.2
0.03																	0	0.3	0.6	0.9	1.3	1.6	1.9
BASEFLOW	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6	9.6
TOTAL	9.6	9.9	10.9	12.8	16.0	20.3	27.1	36.6	50.8	69.9	92.9	118	143	168	191	211	227	235	235	228	217	202	184

Table 6.24A Convolution of unit hydrograph and net rain profile

Step 19

The average non-separated flow per km^2 is calculated from Equation I.6.42

$$ANSF = 0.00033\ (CWI-125)+0.00074RSMD+0.003$$

This is added to the response runoff hydrograph

$ANSF = 0.026$ cumecs/km^2
ie $= 9.6$ cumecs
Thus the 50 year design flow for the example catchment is estimated by the unit hydrograph/loss method as 235 cumecs and the complete hydrograph is also obtained.

ESTIMATION OF THE MAXIMUM FLOOD

The concept of the maximum flood eludes precise definition. It is not the impossible flood; therefore it must have an (infinitesimal) exceedance probability. In the Report, the aim has been to avoid the semantics of definition and to recommend, for practical purposes, a consistent procedure for estimating a discharge (and runoff volume) which can be used as a bound to unrealistic extrapolation of the flood frequency curve. It is important to remember that it is an estimate with an error of estimation. Factors of safety can be introduced with this in mind, but the procedure described does not include them, although several steps are conservative; nor is an overall error of estimation quantified in the Report. It is therefore very important that those using the methods in final design should read the Report carefully and be aware of the errors involved in the separate stages of estimating rainfall, percentage runoff and unit hydrograph parameters.

It is usually necessary to route through a reservoir and therefore to estimate both the peak and the shape of the maximum flood, and the unit hydrograph approach is therefore recommended. In most cases of final spillway design or reappraisal of existing reservoirs, several years of records should be available from the site. These local records must be used to improve estimates of percentage runoff and unit hydrograph parameters derived from the Report's prediction equations. (But note that local revision of maximum rainfall estimates is not recommended.) Once the unit hydrograph estimate has been improved by deriving one for the site from good quality rainfall and runoff data (I.6.8.4), or by estimating the catchment 'lag' from rainfall and river level records (I.6.5.3), it is recommended that, for application to maximum flood estimation, the derived unit hydrograph should be made peakier by reducing the time to peak by a third. With the time to peak (and base length) reduced by a third, all ordinates of the unit hydrograph must be increased by a half to preserve the volume.

This adjustment is recommended for a number of reasons. It is one way of allowing for the effect of storms moving downstream over the catchment, it matches the average ratio of minimum to mean observed times to peak, and it takes account of tests on very large events (I.6.6.3). (Insofar as the adjustment reflects the belief that runoff processes are faster in extreme events, this may not be appropriate for larger catchments with extensive flood plain storage, but this should rarely affect reservoir design.)

The choice of a rainfall profile for maximum flood estimation is particularly difficult. The Report (Volume II) enables maximum point rainfall to be estimated for any duration and location. It is left to the hydrologist to decide on the storm duration and the time distribution within the duration. American practice is to distinguish between 'probable maximum precipitation' which is derived from a depth-duration set of design values based on all storm types and a 'probable maximum storm' being the maximisation and possible transposition of a single storm type. They specifically exclude the possibility of a storm being so structured that it could produce maximum values for all durations and all areas.

It is suggested that this exclusion cannot be applied in this country. Maximum rain intensities observed in the United States are several times greater than ours and there have been occasions in the United Kingdom when relatively severe localised thunderstorm activity has been associated with more general rain. Consequently, it is recommended that in this country a profile should be adopted which contains the estimated maximum fall in every duration centred at the peak of the storm profile.

The probability that an estimated maximum storm could be structured in this way varies with duration and location. It is more likely for storm durations of 12 hours or less in eastern Britain than for longer durations in the west. However, more investigation is needed before such generalisations can be translated into firm recommendations to relax the 'worst profile' assumption in some circumstances. The profile would be symmetrical and, on this basis, the 'critical' duration would be infinite, but for practical purposes it is suggested that the duration, D, be calculated as before (in Section I.6.8.2) and that the effect of the preceding rains be taken into the calculation of a design value for antecedent wetness (CWI).

Although the analysis of records for the areal reduction factor was based on a statistical concept of ARF, it would seem reasonable to use the same ARF for the estimated maximum flood until specific study provides a better alternative.

The addition of snowmelt presents problems. Just as it seems illogical to nest 2 h and 24 h maxima in the same profile, so it seems unlikely that snowmelt (probably winter) should be considered at the same time as a 2 h maximum rainfall (probably summer). But these combinations cannot be ruled out until more work on seasonal maxima has been done. In deciding how much snowmelt to allow, consideration must be given to location but most small catchments in upland Britain can be assumed to be capable of supplying the suggested design rate of 42 mm/day for the whole of the design duration D and preceding 'wetting' period of 2D. It is recommended that this should be added to the rainfall profile as a uniform rate as it seems unreasonable to assume that the profile of snowmelt (controlled by temperature) should exactly coincide with that of the rainstorm.

There may be occasions (large, lowland catchments) when 42 mm/day cannot be maintained for the total 3D period but it can be maintained for 24 hours or less anywhere in the UK. The way in which this is checked is discussed in the Report.

It is necessary to mention the possibility of 'frozen ground'. The March 1947 flood (snowmelt and rain) is commonly believed to have been aggravated by the long spell of cold weather which preceded it and which might have frozen the top layers of soil. If it is thought appropriate to make some allowance for this effect, a convenient way of doing so is to adjust the soil index to its maximum value of 0.5. Whether or not the allowance should be made must remain a matter for the judgement of the engineer. Extreme quantities of snowmelt and rainfall are already being combined and the rainfall is being distributed in time with the worst profile. However, pending a more detailed study of the variation between summer and winter estimated maxima for rainstorms of various durations, it is wise not to regard even the occurrence of an extreme thunderstorm over a frozen catchment with deep snow lying as physically impossible.

When a catchment consists of areas of great disparity, as with an urban area at the downstream end of a chalk catchment, it is advisable to consider the most responsive area alone as well as the catchment as a whole.

As a final point, note that local data are of the utmost importance particularly in the prediction of percentage runoff. The standard error of estimate is ± 15% when using the regression equation but a few years of record are sufficient to replace the fixed component of the prediction equation (I.6.5.8) with a figure derived from local data.

Throughout this account the reader will find subjective words like 'unlikely'. Despite a policy of leaving engineering judgement to the design engineer, a number of arbitrary decisions have had to be made. It follows that engineers must be allowed to exercise additional judgement in specific circumstances; it is hoped that the procedures recommended here provide the framework for them to do so.

Example

The procedure for estimating the maximum flood on the West Lyn at Lynmouth will now be described. Steps 1-9 are worked in the same way as in the previous example. The specific numerical values for these steps are:

Area	=	23.5 km^2
Stream length	=	9.2 km
Stream slope	=	29.7 m/km
SAAR	=	1500 mm
r	=	25%
M5-24 hours/M5-2 day	=	82.1%
M5-2 day	=	85.0 mm
M5-24 hours	=	69.8 mm
M5-1 day	=	62.9 mm
ARF (1 day)	=	0.97
M5-1 day x ARF	=	61.0 mm

SMDBAR	=	6.0 mm
RSMD	=	55.0 mm
URB	=	0.0
URBT	=	1.0
T_p	=	3.1 hours
T_p modified for 'maximum' study (x 0.67)	=	2.1 hours
T	=	0.2 hour
new T_p	=	1.7 hours
D	=	4.2 hours

To avoid confusion with the normal procedure, subsequent step numbers will have the letter M after the number. The calculations for estimated maximum rainfalls of various durations are based on Chapter 4 of Volume II.

Step 10M

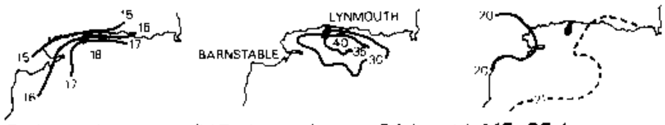
Estimate the catchment average 2 hour and 24 hour maximum point rainfalls from Figures II.4.1 and II.4.2 respectively	Relevant parts reproduced in Figure I.6.67. 2 hour EM 160 mm 24 hour EM 300 mm

Step 11M

Estimate ratio M5-25 days/SAAR from Figure II.3.4. Hence estimate M5-25 days	Figure I.6.67 M5-25 days/SAAR = 20.6% M5- 25 days = 309 mm
From Table II.4.2 (Table I.6.22) extract the relevant growth factor for the estimated maximum 25 day fall	Table I.6.22 Growth factor = 1.92 EM 25 days = 593 mm
From Table II.4.3 find ratio of maximum 192 hours to that for 25 days and hence estimated 192 hour maximum. Relevant parts reproduced as Table I.6.25a	EM192 hours/EM25 days = 0.69 EM192 hours = 409 mm
Also from Table I.6.25a use the given ratios to calculate estimated maxima for durations between 24 and 192 hours	EM48 hours = 336 mm EM72 hours = 351 mm EM96 hours = 366 mm

From Table II.4.1 (Table I.6.25b) calculate estimated maxima for durations less than 2 hours

EM10	min	=	56.0 mm
EM15	min	=	73.6 mm
EM30	min	=	102.4 mm
EM60	min	=	129.6 mm



(a) Estimated max. 2-hr rainfall, 10's of mm   (b) Estimated max. 24-hr rainfall, 10's of mm   (c) $\frac{\text{M5-25day}}{\text{SAAR}} \times 100$

Figure 6.67

Climatic parameters extracted from folded maps - River West Lyn

Table 6.25   Estimated rainfall maxima for various durations

Average annual rainfall (hundreds of mm)	Ratio 192 hours/25 days	(a) Durations ≥ 24 hours Ratio of stated duration (hours) to 24 hours			(b) Durations < 2 hours % of stated duration (min) to 2 hour						
		48	72	96	1	2	5	10	15	30	60
5–6	0.84	1.10	1.13	1.17							
6–8	0.80	1.10	1.13	1.17							
8–10	0.76	1.10	1.14	1.18	6	11	23	36	47	65	83
10–14	0.71	1.11	1.16	1.20							
14–20	0.68	1.12	1.18	1.24	6	11	22	34	45	62	79
20–28	0.65	1.14	1.23	1.32							
28–40	0.62	1.20	1.31	1.42	6	10	21	32	43	59	75
40	0.60	1.23	1.35	1.48							

Finally plot the amount for various durations on double log paper. For durations between 2 and 24 hours a straight line is suggested. For the longer and shorter durations a smooth curve should be drawn

Figure I.6.68 shows the estimated maximum point rainfall for various durations on the Lynmouth catchment

For each duration, a different areal reduction factor will apply (Figure II.5.1 or Figure I.6.58)
The areal rainfall (P) for the design duration is read from the figure

Catchment average estimates of the maxima are shown in the lower line of Figure I.6.68

P for 4.2 hours = 180 mm

(This full procedure is, of course, unnecessary on any particular catchment; a duration of either 25 days or ;0 min could be relevant, but not both. It is illustrated here only for the sake of completeness)

The snowmelt allowance may be added at this stage

At 42 mm/day, the total snowmelt in the design duration amounts to 7 mm. Total design precipitation = 187 mm

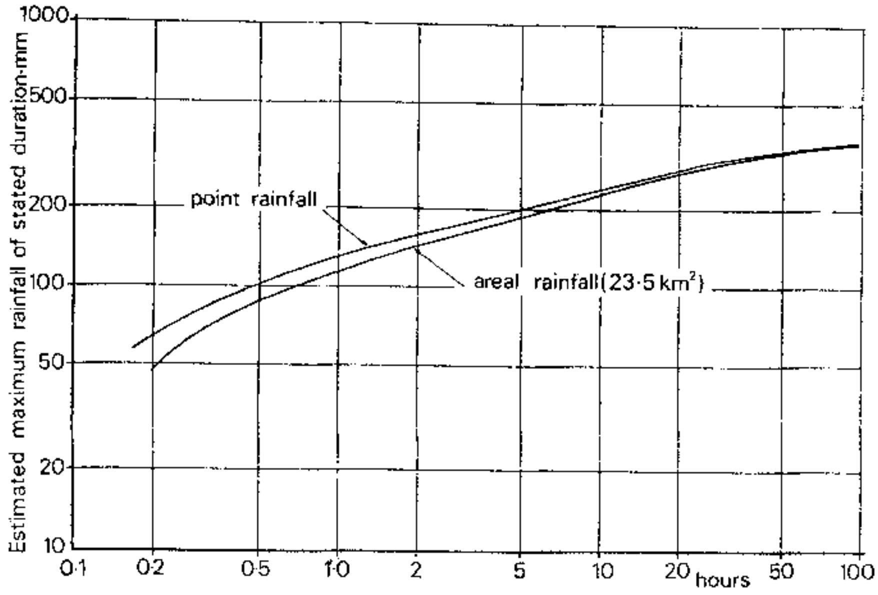


Figure 6.68 Estimated maximum falls for various durations - River West Lyn

Step 12M

The design value for CWI is based on the assumption that the D hour storm occurs in the middle of the 5D hour storm.

It is therefore assumed that half the difference between the D hour and 5D maxima may fall in 2D hours preceding the design storm

The CWI at the beginning of the 5D hour period is taken as 125. The rain plus snowmelt (P_a) falling in the next 2D hours is assumed to be uniform (rectangular profile) and the design CWI can be approximated as

$125 + P_a \times 0.5^{D/24}$

Steps 13 and 14 are the same as described in Section I.6.8.2 and above. SOIL would be increased to 0.5 if 'frozen ground' was to be considered

The estimated maximum for 5D = 21 hours is read from Figure 6.68

EM 21 hours = 280 mm

50 mm in 8.4 hours

The snowmelt allowance is added = 14 mm in 8.4 hours

Thus total antecedent rainfall plus snow melt P_a = 64 mm

CWI = 182

$S_1 = 0.0$, $S_2 = 0.6$, $S_3 = 0.0$, $S_4 = 0.0$, $S_5 = 0.4$, $S_u = 0.0$,

SOIL = 0.38

SPR = 36.3%

PR = 66.5%

In this example frozen ground has not been considered. If the soil index had been increased, the percentage runoff would be 78% rather than 66.5% and the peak flow would be about 330 cumecs.

Step 15 does not apply.

Step 16M

The estimated maximum rainfall is to be symmetrically distributed according to the extreme profile such that the estimated maximum occurs in every duration centred on the peak of the storm profile

Referring to Figure 6.68, there are
48 mm in a duration of T (0.2) hours;
95 mm in a total duration of 3T (0.6) hours;
114 mm in 5T (1.0) hours etc.

The hyetograph is built up with 48 mm in the central 0.2 hours, with 95 mm total in the central 0.6 hours, giving (95-48)/2 mm in the two intervals on either side of the centre, and so on. The resulting hyetograph is shown in Figure I.6.69, the net rain hyetograph is also shown

The snowmelt allowance is added as a uniform rate

Snowmelt allowance 0.33 mm/0.2 hour

Steps 17, 18 and 19 are the same as described in Section I.6.8.2 and above.

Q_p = 129.4 cumecs/100 km^2

TB = 4.29 hours
ANSF = 0.060 cumecs/km^2
= 1.4 cumecs

Hydrograph shown in Figure I.6.69

Peak = 280 cumecs
(estimated maximum flow, River West Lyn at Lynmouth).

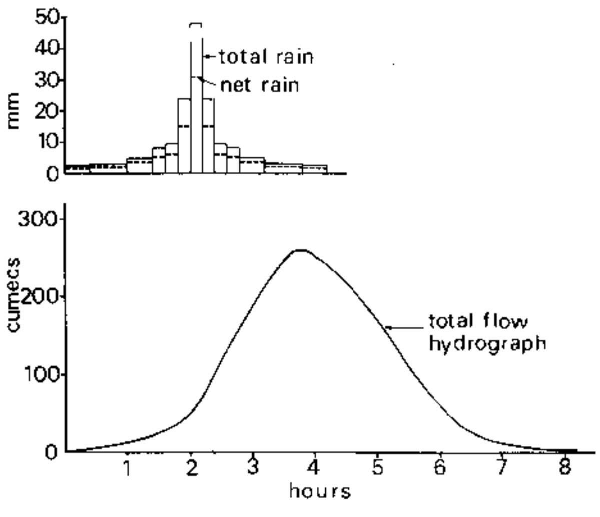


Figure 6.69 The estimated maximum event - River West Lyn

RAPID METHOD OF ESTIMATING THE MAXIMUM FLOOD

The previous estimate may be compared with the answer provided by a rapid prediction equation described by Farquharson, Lowing and Sutcliffe, (Some aspects of design flood estimation, BNCOLD Symposium, Newcastle, September, 1975), which may be useful for preliminary screening.

$$\begin{aligned} \text{EMF} &= 0.835\text{AREA}^{0.878}\text{RSMD}^{0.724}\text{SOIL}^{0.533}(1+\text{URBAN})^{1.308}\text{S1085}^{0.162} \\ &= 0.835\ 23.5^{0.878}\ 55^{0.724}\ 0.38^{0.533}(1)^{1.308}\ 29.7^{0.162} \\ &= 0.835 \times 15.9881 \times 18.198 \times 0.5971 \times 1 \times 1.7322 \\ &= 251 \text{ cumecs} \end{aligned}$$

This formula was based on estimates of the maximum flow for 80 catchments by the unit hydrograph technique, using observed values for the unit hydrograph and standard percentage runoff. These estimates were then related to catchment characteristics to give a prediction formula; this formula gives a rapid estimate of the maximum flood, but lacks the precision of the direct application of the unit hydrograph approach.

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ACKNOWLEDGEMENTS

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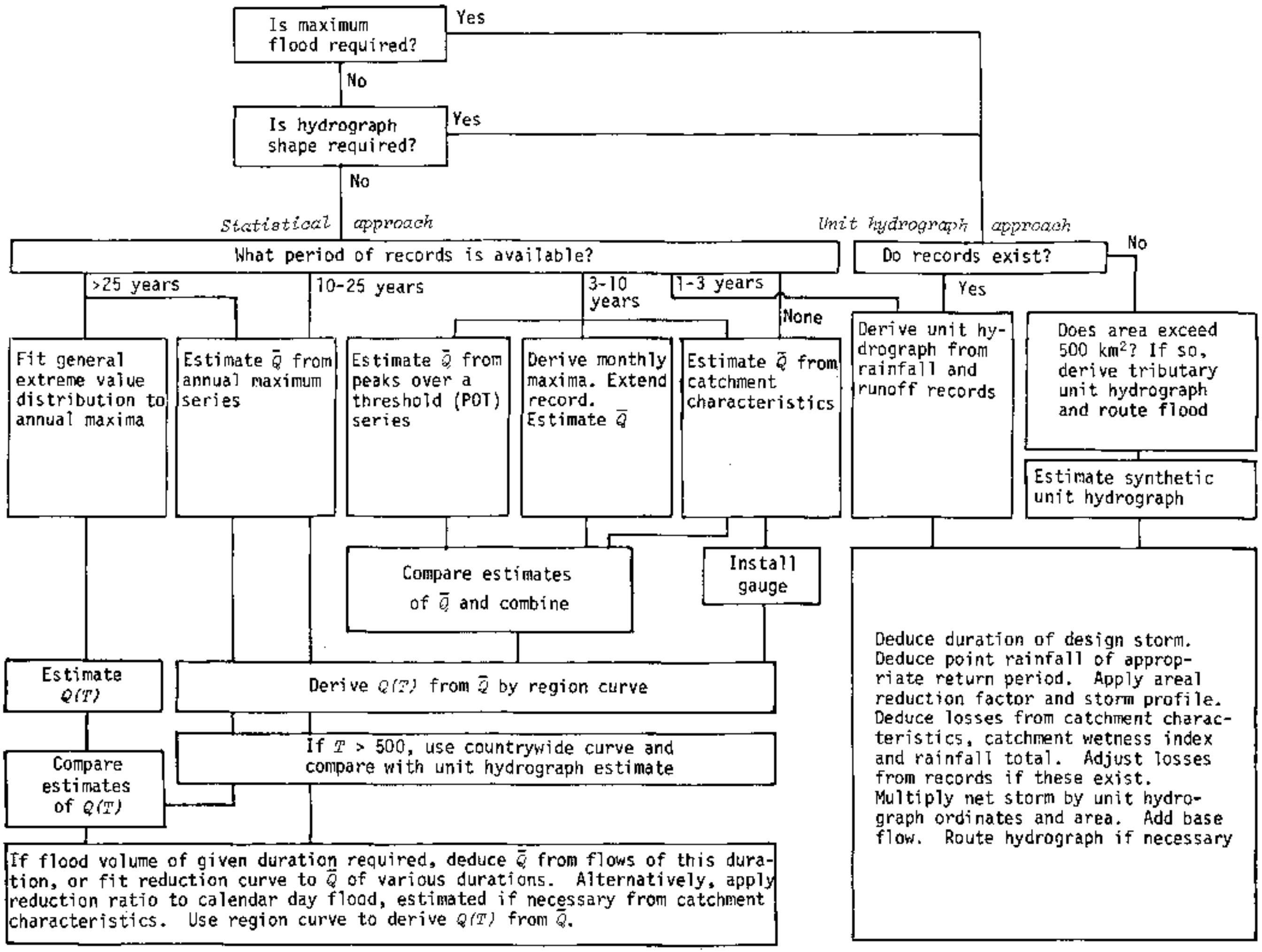


FIGURE A.1 ESTIMATION OF DESIGN FLOOD

APPENDIX A

CATCHMENT CHARACTERISTICS

The assessment of floods at ungauged sites relies on the incorporation of data on certain catchment characteristics into the relevant formulae described earlier in the Guide. This appendix describes the characteristics used and their estimation.

The catchment characteristics and notations used are:

	Notations	Range of values at stations used in study: Minimum	Maximum	
Area	AREA	0 038	9868	km^2
Stream length	MSL	0.27	238.75	km (excluding Irish stations)
Stream slope	S1085	0.19	117.78	m/km
Stream frequency	STMFRQ	0.01	7.54	Junctions/km^2
Soil index	SOIL	0.15	0.50	
Lake index	LAKE	0.000	1.000	
Urban development	URBAN	0.000	0.808	
Annual average rainfall	SAAR	551	3454	mm
Net 1-day rainfall of 5-year recurrence	RSMD	15.6	117.5	mm

The range of values found at the stations used in the study indicates the likely magnitude of these catchment characteristics. Values near or beyond these limits should be checked for arithmetic errors.

It should be borne in mind when making estimates for ungauged catchments that regression equations are more precise in the middle of the range of data on which they are based. Unfortunately, the study has been able to draw on few records from small catchments, from the north of Scotland, from Northern Ireland, or from urban catchments. However, an important argument for a countrywide study is that it uses the widest possible range of records. The fact that the regression equation for the mean annual flood, for instance, is compatible with our physical knowledge of floods suggests that it is preferable to use the countrywide equations and if necessary adjust for a consistent bias in local records rather than rely on local analysis alone.

There are two instances where the lack of data suggest a separate approach for ungauged sites. Less than 5% of the records had an urban fraction over 0.25, and less than 10% had a lake index over 0.33. Estimates for ungauged sites with an urban fraction over about 0.25 should be based on urban drainage design methods, while estimates for catchments with a large proportion draining through a lake or reservoir should be adjusted by reservoir routing techniques.

AREA

The area draining to a site is the most fundamental catchment characteristic. It should be measured at an early stage in any study by planimetry or by counting squares from a map on which the watershed is drawn. Do not overlook any diversions or leats which increase or decrease the flood-producing area from the topographical catchment. The Ordnance Survey First Series 1:25 000 map should be used for deriving topographic characteristics as this was the series used during the investigation. Regressions will be revised or a conversion factor provided when the Second Series is complete.

Example: the River Almond at Craigie Hall (Grid reference NT 165752)

AREA = 369 km^2

STREAM LENGTH (MSL) AND STREAM SLOPE (S1085)

Main stream length (MSL or L) is derived during the assessment of stream slope (S1085). The stream slope is that defined by the United States Geological Survey as the mainstream slope between the 10 and 85 percentiles of mainstream length (upstream from the gauging station).

Raw data for both S1085 and MSL are derived as follows. Choose the main stream from maps which in most cases is simply the longest stream in the basin. In cases of difficulty, work upstream and at every junction follow the stream draining the larger area. Distances are measured upstream from the station with precision dividers set at 0.1 km (4 mm on the 1:25 000 map). It helps to mark every fifth step. Once the total length to the end of the stream is known, the lengths and elevations of the 10% and 85% points are used to calculate slope; the units used are parts per thousand or metres per kilometre. Stream length is sometimes described as that 'along the main channel between the gauge and the divide' which implies the length to the watershed. In the Flood Studies the channel was defined as the blue line on the 1:25 000 map. The variation in S1085 values is shown in Figure I.4.4 of the Report, p. I.299, which may help to prevent gross errors in calculation.

Example:

Number of steps (N) = 446; therefore MSL, the stream length = 44.6 km;

Mark the points 0.1 N and 0.85 N steps upstream from the starting point.

By interpolation between contours,
elevation at 0.85 N steps upstream = 625 ft
and at 0.1 N steps upstream = 90 ft
Difference = ΔH
= 535 ft = 163. 068 m

S1085 = $\Delta H/0.75$ MSL
= 163.068/33.45 m/km
= 4.87 m/km

As a precaution against inaccurate setting of dividers, it is suggested that 50 steps be made along a straight line before and after map measurement and any departure from the expected 200 mm allowed for by a correction factor.

STREAM FREQUENCY (STMFRQ)

The channel network is described by 'stream frequency' simply measured by counting channel junctions on the 1: 25 000 First Series maps and dividing by basin area. The precise technique is as follows.

Once the necessary maps are assembled in a logical order, the stations or sites for estimation are marked to avoid duplication if more than one site is to be measured. The number of natural stream junctions is counted upstream from the lowest site, which is also included as a junction. It is best to work progressively up each tributary; the running total is noted at each major junction and at additional gauges. Artificial channels in fenland or flood plains and also canals are ignored. Where natural channels exist, but are not shown on the map, for instance in urban areas, or where junctions occur in a lake or reservoir, the missing junctions are counted.

In catchments under 0.2 km^2, the following procedure should be adopted to avoid exaggerated estimates. Move downstream to the nearest third order stream (see Figure I.4.5) and measure the stream frequency of its basin. Figure I.4.7 can be used as a rough check on the stream frequencies obtained by the user of the Report.

<u>Example</u>

$$\begin{aligned} N &= 375 \text{ junctions} \\ AREA &= 369 \text{ km}^2 \\ STMFRQ &= N/AREA \\ &= 1.02 \text{ junctions/km}^2 \end{aligned}$$

SOIL INDEX (SOIL)

The soil index is based on the soil map (Figure I.4.18) given in the Report, where five classes of soil are shown based on their 'winter rain acceptance potential'. Weights were ascribed to each soil class which indicate their individual runoff potential; a soil index for a catchment is derived by measuring the fractions of the catchment within each soil class, and adopting a weighted mean of these soil fractions (S_1, S_2, ...)

$$SOIL = \frac{0.15S_1 + 0.30S_2 + 0.40S_3 + 0.45S_4 + 0.50S_5}{S_1 + S_2 + S_3 + S_4 + S_5}$$

The areas of each class are determined by overlaying the soil map with a catchment map at the 1:625 000 scale. Sufficient accuracy is normally obtained by counting the squares on 1/10th inch graph paper.

Example

	Soil Class		No. of squares
	1	(N_1)	$0 \times 0.15 = 0$
	2	(N_2)	$0 \times 0.30 = 0$
	3	(N_3)	$0 \times 0.40 = 0$
	4	(N_4)	$119 \times 0.45 = 53.5$
	5	(N_5)	$28 \times 0.50 = 14.0$
Unclassified		(N_u)	0 Total (W) 67.5

$$N_s = N_1 + N_2 + N_3 + N_4 + N_5 = 147$$

$$\text{SOIL} = W/N_s$$

$$= \underline{0.459}$$

As a useful check, the soil index must lie in the range 0.15 to 0.50. A check on the catchment area is given by $(N_s + N_u) \times 2.52$.

LAKE INDEX (LAKE) AND URBAN INDEX (URBAN)

An index of lake storage and an index of urban development are used in some of the equations for predicting floods at ungauged sites. These indices are no substitute for lake or reservoir routing where the design site is immediately downstream of large storage, or for using urban runoff models where the flood runoff from a predominantly urban area is required. However, where the area draining through a lake or from urban development is not too high a proportion of the catchment area, results are improved by taking these into account rather than ignoring them.

All the lakes or reservoirs whose surface areas are less than 1% of the area contributing to that lake are ignored. In practice, each tributary is followed until a lake or reservoir is met whose area is greater than 1% of the area contributing; the contributing area is then recorded. It is unnecessary to continue upstream as a reservoir within this contributing area does not count. This is repeated on all other tributaries within the gauged catchment and all contributing areas are summed to give the total area contributing to lakes or reservoirs. This total contributing area is divided by the total area of the gauged catchment to give a lake index (LAKE). This index has the disadvantage, when used in a multiplicative equation, that the effect of increasing the lake fed fraction from 0.01 to 0.1 is the same as from 0.1 to 1.0. To overcome this and to provide an index which vanishes as a product when no lakes are present, the index is transformed from LAKE to (1 + LAKE) when used in regression analysis.

Examples

The surface area of two reservoirs in the Almond catchment exceed 1% of their respective catchment areas which total 14.8 km^2 or 0.040 of the total catchment area, therefore

$$\text{LAKE} = 0.04$$

Urban fraction is estimated from the area shown as built up on a suitable scale map, e.g. 1:63 360. This was estimated as 42.0 km^2 or 0.114 of the catchment, therefore

URBAN = 0.114

ANNUAL RAINFALL AND SHORT-TERM RAINFALL INDEX (SAAR & RSMD)

The standard annual average rainfall (SAAR), and the 1 day rainfall of 5 year return period minus the effective mean soil moisture deficit, (RSMD), are used as indices of catchment rainfall. The annual rainfall is an index of climate, while the net short-term rainfall is an index of flood-producing rainfall; in practice the two were found to be fairly closely related.

The annual average rainfall (SAAR) is obtained from Figure II.3.1. The average for the catchment may be obtained by sampling at about 20 points equally spaced on a grid overlay and taking the arithmetic mean. Alternatively, the weighted areas technique can be used.

SAAR = 914 mm

The calculation of RSMD requires M5-2 day rainfall from Figure II.3.2, the ratio r (M5-60 min/M5-2 day) from Figure II.3.5, and the effective mean SMD (SMDBAR) from Figure I.4.19. In all three cases, catchment average values are required and these may be obtained by grid point sampling or weighted areas.

The ratio M5-24 hours/M5-2 day is determined in terms of r, from Table II.3.7 (reproduced as Table I.6.21). M5-24 hours follows and this is converted to M5-1 day by dividing by 1.11 (Table II.3.1). The 1 day areal reduction factor (ARF) is obtained from Figure II.5.1 reproduced as Figure I.6.58 in the unit hydrograph example (Section 4). The 1 day ARF depends only on area as follows:

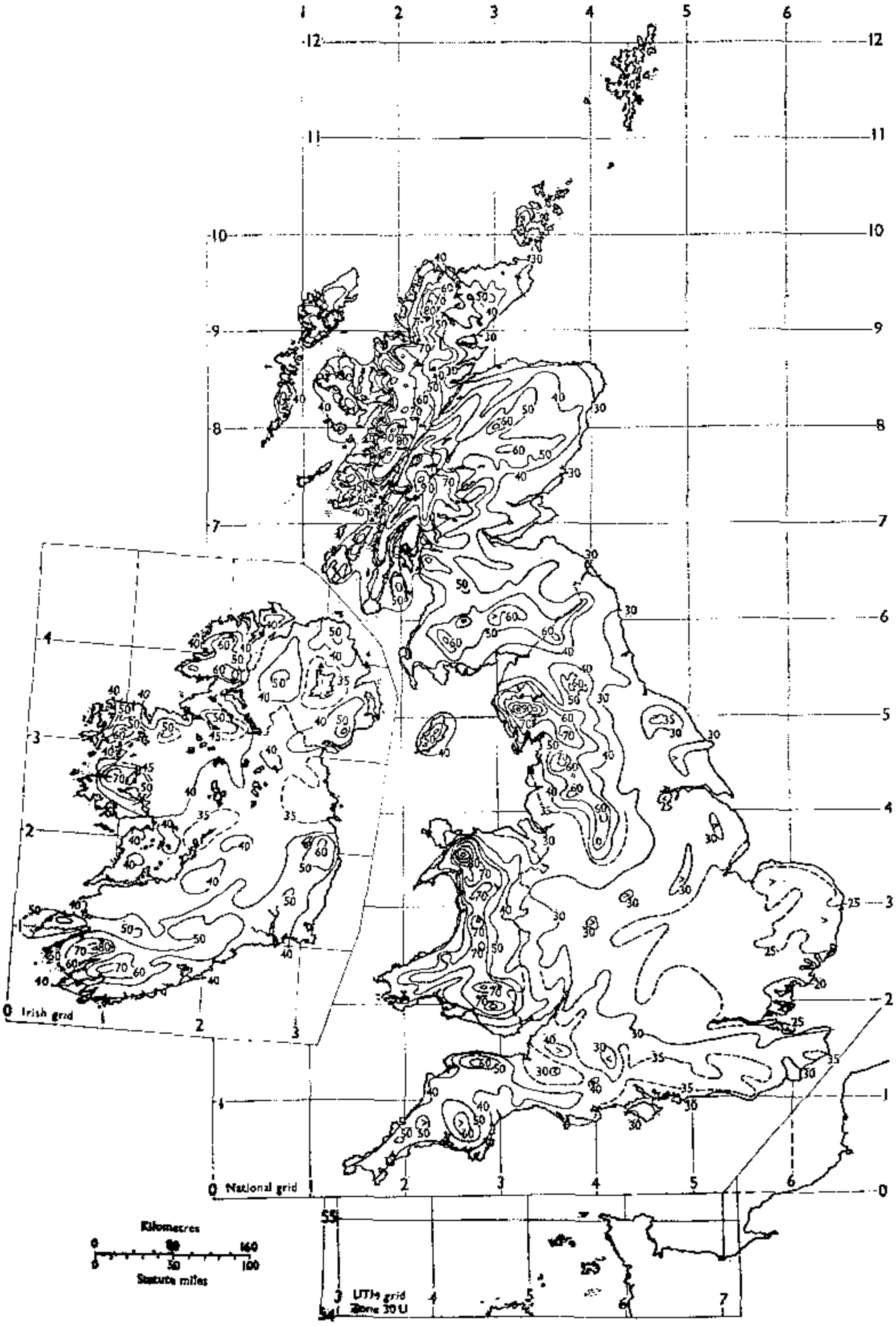
Area (km^2)	5	10	20	50	100	200	500	1000
ARF	0.980	0.975	0.970	0.955	0.940	0.925	0.910	0.880

Example

RSMD = M5-1 day x ARF - SMDBAR

r	=	25%
M5-24 hour/M5-2 day	=	82%
M5-2 day	=	57 mm
M5-24 hour	=	46.7 mm
M5-1 day	=	42.0 mm
ARF (1 day)	=	0.92
M5-1 day x ARF	=	38.6 mm
SMDBAR	=	6.6 mm
RSMD	=	32 mm

The relationship between RSMD and SAAR (standard annual average rainfall) as shown in Figure I.6.59 allows an initial estimate of RSMD to be made, as does the outline map of RSMD published in the ICE Flood Studies Conference 1975, p. 104, and reproduced overleaf.



Outline map of RSMD, the net 1 day rainfall of 5 year return period: values in millimetres, < denotes minimum area, > denotes maximum area

APPENDIX B

SUMMARY OF FLOOD STUDIES REPORT

This appendix summarises the background and main conclusions of the Flood Studies Report, and is included to give readers who have not had the time to study the main report some knowledge of its contents.

Background

The Institution of Civil Engineers' Committee on Floods recommended in 1967 that a new investigation should examine all aspects of flood hydrology; meteorological records should be studied to understand the causes of floods, to extend flood records, to assist in flood frequency analyses and to provide estimates of probable maximum precipitation. All available flood records should be assembled and reviewed; frequency analyses of flood peaks and volumes should be carried out; regional analyses and correlations with catchment characteristics should be undertaken to improve single station frequency distributions and to estimate flood frequencies at ungauged sites; unit hydrographs, soil infiltration characteristics and snowmelt should be studied to derive precipitation-runoff models for use with the results of meteorological studies; flood routing techniques should be reviewed and tested.

The Report describes this work and presents methods of flood estimation based on analysis of flood records from the British Isles. As flood estimates are required for the design and economic appraisal of a variety of engineering works, including dam spillways, bridges and flood protection works, the objective was to derive methods of estimating the flood to be exceeded at a given site on average once in T years (the flood of T year return period) and also the estimated maximum flood. Two main approaches were used in the study; the first was based on statistical analysis of flood series at all gauging stations, and the second on investigation of rainfall and resulting runoff at selected stations. In practice, estimates are often required at sites without records, so the results of each of these investigations have been related to catchment characteristics.

The five volumes of the report are summarised below; the meteorological studies (Volume II) are described first as the results are required for an understanding of the hydrological studies.

METEOROLOGICAL STUDIES - VOLUME II

The meteorological study provides estimates of the rainfall depth corresponding to a given duration and return period, both at a point and over an area, together with a profile or time distribution of this rainfall. The rainfall records analysed in this study comprised daily falls from 600 long-term stations with an average record of 60 years, 6000 additional stations for the decade 1961-1970, and also records from some 200 autographic raingauge stations. Rainfall durations of 2 days and of 60 minutes were used as a basis for analysis and other durations were related to these basic periods.

For each station the highest 2 day rainfalls recorded in each year provided an annual maximum series. The 2 day rainfall of 5 year return period (2 day M5), or the rainfall exceeded on average once in five years, was estimated from the mean of the upper two quartiles of the annual maxima. To derive the corresponding rainfall MT of T year return period, stations within various ranges of M5 were grouped to give curves of MT/M5 or 'growth factor'. Similar growth curves were deduced for other durations and ranges of M5; it was found that for a given M5 a single growth curve could be used for England and Wales to give the ratio MT/M5, regardless of duration. A second curve was derived for Scotland and Northern Ireland.

Because the rainfall MT of any return period T years could be deduced from an estimate of M5, the latter was mapped for durations of 2 days and of 60 minutes. Rainfalls of other durations can be deduced from these estimates to give a corresponding M5; rainfall estimates for other return periods can be deduced from the growth curve as a function of M5.

Estimates of maximum 2 hour rainfall were based on major storms adjusted for maximum observed storm efficiency together with maps of precipitable water corresponding to surface dew points. Estimated maximum falls for 24 hours based on maximum storm efficiencies were related to the 2 day rainfall map to provide a map of estimated maximum 24 hour rainfall.

The areal reduction factor used in this report is the ratio between the areal rainfall for a catchment of a given size and the point rainfall of the same duration and return period. It is important to note that this ratio of the magnitudes of two design storms of a given return period is a statistical factor; it should not be thought of as describing the area-magnitude profile of any individual storm or even an average storm. This areal reduction factor was found to be related to duration and catchment size but was not found to vary with location.

Time profiles were extracted for a large number of storms and were centred on the most intense part of the storm. These standardised storm profiles were found not to vary with duration, return period or location to a significant extent. Summer and winter profiles were classified by peak intensity and the probability of various profiles was determined.

A preliminary attempt to estimate rare snowmelt rates was based on the collection and analysis of records of snow depth, snow density and temperatures over snow.

Chapter II.8 gives a practical guide to the estimation of the catchment rainfall for a given duration and return period, a likely time profile for this rainfall and also the maximum rainfall in the same duration.

HYDROLOGICAL STUDIES - VOLUME I

Statistics and flood frequency analysis - Chapters 1-2:

To provide a statistical framework within which floods can be analysed, the properties of the various distributions which have been proposed in flood hydrology are described; by assembling these, the relationships between different distributions are stressed. Methods of fitting distributions to sample data are explained including graphical fitting and fitting by moments and maximum likelihood. The use of correct plotting position affects some methods of fitting and the correct plotting position for different distributions is discussed. Standard errors of estimates are derived for certain distributions.

Certain distributions have been used for annual maximum floods on empirical grounds - for example, lognormal and Pearson Type 3; the extreme value distributions applied by Gumbel have some theoretical basis but still require testing. Because there is no firm theoretical basis for choice between distributions, goodness of fit tests are often used for comparison. However, even this comparison depends on the index of fit and on the plotting positions used. As in earlier investigations, the three parameter distributions were found to be better or more flexible than the two parameter distributions. However, no clear cut choice results from this comparison.

The question of sampling error was considered and theoretical estimates were checked against empirical estimates derived from long term records analysed by decades. A single formula is suggested for the standard error of Q(T) (the flood of T-year return period) regardless of distribution fitted.

On the assumption that the records of an area provide an average picture of the variation of floods, region curves were built up by combining sets of stations in regions and countrywide. Historical records were included in these curves which show an increase in CV (standard deviation ÷ mean) and skewness from the north-west to the south-east of the country.

An alternative to the estimation of floods from the annual maximum series is the use of the series of peaks over a threshold with an assumed distribution. The relative efficiencies of the two methods are discussed.

Methods of including historical records or missing peaks in flood estimation are described; these methods are based on the statistical theory of censored samples.

Extension of short records - Chapter 3:

In many practical cases estimates of the flood regime have to be made from short periods of records even though the sampling error of a short record is bound to be high. The estimates may therefore be improved by adding further information from long term records through extending the short term record by correlation.

A detailed study was made of this technique and a method was developed and used for adjusting flood estimates. It was decided to concentrate on improving the estimate of the mean annual flood and correlation of monthly maxima was used for this purpose. Programs for handling these data on a large scale provided information on the records available within 100 km of the short term station. Correlation maps based on this information enabled long term stations to be chosen to extend batches of short term records. Although the improvement using extended data in regression of flood estimates on catchment characteristics was small, this may be due to the limitations of the regression model. When an individual short term record is to be extended, more detailed attention can be given; an example is presented of the technique which should be adopted in practice, particularly when a short term record covers a period which is known to be biassed.

Flood estimation from catchment characteristics - Chapter 4:

Where no records are available at a site, a preliminary estimate may be made from relations between floods and catchment characteristics. A number of these characteristics were chosen for testing and were measured for those catchments where mean annual flood estimates were available. These characteristics were chosen to be hydrologically relevant, to be as uncorrelated as possible, and to be capable of being measured simply for a large number of catchments.

Physical characteristics which were measured from maps included area and main stream length, channel slope and stream frequency or the number of junctions per unit area.

A soil variable was developed by classifying soils according to winter rain acceptance. Maps showing this classification were used to measure the fractions of catchments in each soil class and these were used as the basis of a single soil index. The fractions of each catchment draining through lakes or under urban development also provided useful indices.

While annual average rainfall has been used previously as an index of climate, it was considered desirable to test short term rainfall and, if possible, rainfall excess or RSMD, defined as the 5 year return period value of the daily rainfall minus soil moisture deficit. It was found that the difference between rainfall and rainfall excess did not vary greatly with return period and this difference could be estimated as an effective mean soil moisture deficit. This was mapped and provided RSMD from short term rainfall maps.

The mean annual flood and the coefficient of variation (CV) were chosen for regression on catchment characteristics; these were estimated from the annual maximum series. Alternative estimates based on the series of peaks over a threshold proved interchangeable and both catchment characteristics and flood statistics are given in the Master List in Volume IV, Chapter 5.

Regressions of mean annual flood on catchment characteristics were based on all stations of grades A-D with records over 5 years, extended by correlation where possible. The best estimate of mean annual flood was given by regression on area, stream frequency, stream slope, soil index, net short-term rainfall, and a lake index.

It was found that little would be gained by dividing the records into ranges of catchment size; on the other hand, significant improvements could be made by dividing the country into geographical regions. The best preliminary estimates of the mean annual flood could be obtained by deriving common regression coefficients and different intercepts for groups of regions. The Thames, Lee and Essex region proved an exception in that different variables and coefficients were required.

It proved impossible to relate much of the variation of CV to catchment characteristics so that region curves have to be used for ungauged sites. A small proportion of the variation of CV could be attributed to climate and this is consistent with the growth curves of the meteorological study and with the regional mean CV values.

Estimation of flood volumes - Chapter 5:

It was found that the relation of the mean annual flood to duration can be described by a reduction curve which can be fitted to actual records and is particularly useful for durations over one day. However, the chosen parameters of these reduction curves were not well related to catchment characteristics.

On the other hand, simple ratios of 3 day and 10 day floods to 1 day floods were well related to channel slope and thus can be used to estimate flood volumes of various durations from calendar day floods. The daily floods can in turn be estimated from catchment characteristics or deduced from records.

Synthesis of the design flood hydrograph - Chapter 6:

In parallel with the statistical analysis of flood records, an investigation of catchment response to rainfall was carried out. If an estimated rainfall of a given duration and return period can be converted into runoff, an alternative estimate of the flood of a given return period can be deduced with not only the peak but also the shape of the flood specified. Unit hydrograph techniques allow the problems of predicting runoff volume and timing to be separated by isolating the quick response component of the runoff hydrograph.

The investigation of these prediction methods was based on records of about 1500 rainfall/runoff events on 140 catchments. Consistent rules for hydrograph separation provided estimates of response runoff and made it possible to compare runoff volumes with rainfall amounts and antecedent conditions on these catchments. These comparisons were used in a combined statistical analysis to provide a means of predicting the percentage response runoff in terms of soils and urban fraction, and of antecedent conditions and rainfall amount.

The loss, or the difference between rainfall and response runoff, was distributed through the storm on the assumption that the loss varied inversely with a catchment wetness index (CWI). A comparison of the resulting net rainfall and the response runoff was used to derive a least squares estimate of the unit hydrograph, defined as the response to 10 mm of net rainfall falling in 1 hour. A linear approximation to this unit hydrograph was specified by the time to peak, peak flow, and the width at half the peak, where these three dimensions were highly correlated. No apparent tendency was found within the data for these to vary consistently with runoff volume or rainfall intensity; therefore the average for each dimension was used to represent the catchment in subsequent statistical analysis.

The time to peak was found to be related to channel slope and other factors, with evidence of the effect of urban development. Once the time to peak has been estimated, the remaining dimensions of the unit hydrograph can be directly deduced from it.

Prediction of the runoff volume, together with a prediction of its time distribution, does not provide a direct means of estimating the flood of a given return period; the flood of a given magnitude could result from a number of combinations of different rainfall depths, durations and profiles with different antecedent conditions. A study has shown that it is possible to derive the flood distribution by sampling these various combinations. This approach can be reduced to the choice of a single value for each variable - rainfall (depth, duration and profile) and antecedent condition - to derive the design flood.

This technique offers a gradation of prediction methods according to the information available. If no records exist, the hydrograph can be estimated from catchment characteristics alone. If some records exist, the time to peak can be deduced from the measured lag time and the remaining unit hydrograph dimensions can be derived from this. With more evidence or further analysis, the actual unit hydrograph can be derived and substituted for a synthetic one. From evidence of actual rainfall/runoff events, the runoff volume prediction could be adjusted for any systematic error in the prediction equation for the particular catchment. Either a standard CWI, a storm duration and profile could be used, or a simulation study could be carried out to deduce the flood frequency relationship from the rainfall frequency.

The application of this technique to estimating the probable maximum flood requires more conservative assumptions about the antecedent condition, storm profile and unit hydrograph. It is suggested that the profile and catchment wetness index at the start of the design duration should be based on the assumption that the estimated maximum rainfall occurs in all durations centred on the storm peak. The unit hydrograph should be adjusted for exceptional conditions by reducing its time to peak and increasing its peak.

FLOOD ROUTING STUDIES - VOLUME III

An investigation of the suitability of existing flood routing methods for British rivers was combined with the development of a new method for routing floods in rivers with extensive flood plains.

The simple storage-routing methods, such as the Muskingum method, are sufficiently accurate to route inbank floods; the diffusion method also gives accurate results. The formula developed by Forchheimer for the attenuation along the reach of the peak level or discharge for a flood can be used if the predicted attenuation is less than 10% of the original peak discharge.

When there is a flood plain associated with the river, the values of the parameters used by these methods for an overbank flood can be considerably different from the values of the same parameters for an inbank flood in the same river. A new flood routing method presented in this volume uses a parameter which allows the speed of the flood wave to vary with discharge and a second parameter which describes the effect on a flood wave of the irregularities in the channel geometry.

If an onstream reservoir is to be sited well downstream in a river system, this method can be used to route a flood hydrograph from an upstream section to the reservoir site.

HYDROLOGICAL DATA - VOLUME IV

The basic hydrological data collected and used during the investigation are presented or summarised. These data comprise lists of gauging stations and their gradings and catchment characteristics, flood statistics, the basic peak flow records for 530 stations, with historical records where these were found, and summaries of 1500 events used in catchment response studies.

MAPS - VOLUME V

The large scale maps required to apply the techniques to flood estimating in the British Isles are contained in a separate volume. These include maps of annual average rainfall, 2 day rainfall of 5 year return period, 1 hour 5 year rainfall expressed as a percentage of 2 day rainfall, and estimated maximum 2 hour and 24 hour rainfall. They include a soil map defined in terms of winter rain acceptance potential, estimated mean soil moisture deficit, details of all the river gauging stations used in the analysis with statistics of mean annual flood divided by area, coefficient of variation of annual flood, and residuals from prediction equations for mean annual flood. These maps cover both Great Britain and Ireland.

APPENDIX C

GLOSSARY

A, AREA	catchment area (km^2)
ANSF	average non-separated flow, cumecs/km^2
ARF	areal reduction factor (the ratio of the areal rainfall to the point rainfall of the same duration and return period)
AR3	mean annual 3 day flood/mean annual calendar day flood = $\bar{Q}(3)$/CALMAF, and similarly for AR10
CV	coefficient of variation (usually of annual maximum flood series)
CALMAF	mean of annual maximum calendar day flood series
CWI	catchment wetness index based on antecedent precipitation index (API5) and estimated soil moisture deficit
D	duration (hours)
EV	extreme value, the largest of many observations
EV1, EV2, EV3	related extreme value Type 1, Type 2 and Type 3 distributions used to describe annual maximum floods
F_i, $i = 1, 2, \ldots N$	probability assigned to the lowest, second lowest and so on of a series of observations
F(Q)	distribution function, the probability that a value from the distribution is less than Q
f(Q)	probability density function, the relative probability of a value of Q occurring
g	skewness
GEV	general extreme value distribution, which takes the form of EV1, EV2 or EV3 depending on a shape parameter
k	shape parameter of general extreme value distribution
LAG	time in hours from the centroid of the rain profile to the peak runoff or to a 'centroid of peaks' if more than one peak
LAKE	fraction of catchment draining through a lake or reservoir
ln	natural logarithm
log	logarithm to base 10
MAF	mean of annual maximum instantaneous flood series ($=\bar{Q}$)
M2, M5, ...MT	the rainfall with return period 2, 5,....T years, derived from the series of annual maxima
MSL	main stream length
MT/M5	ratio of the once in T years rainfall to the once in 5 years rainfall, known as the growth factor
M52D	2 day rainfall of 5 year return period
N	the number of years of record (or the number of annual maxima)
P	rainfall

PERC	percentage runoff
POT	peaks over a threshold, eg POT series
Q,q	flood peak discharge
q_o	threshold flow in POT model
Q_p	peak of T hour unit hydrograph expressed in cumecs/ 100 km^2
Q(T), Q_T	flood peak of return period T, the T year flood
$\bar{Q}$	mean of annual maximum instantaneous flood series, the mean annual flood
Q_{max}	maximum flood on record
r	the ratio of 60 minute M5/2 day M5
R	coefficient of multiple correlation
RSMD	one day rainfall of 5 year return period less effective mean soil moisture deficit
S1085	10-85% stream slope (m/km)
SAAR	standard (1916-50) annual average rainfall (mm)
se(.)	standard error
SMD	soil moisture deficit (estimated by Meteorological Office assuming standard catchment)
SMDBAR	effective mean soil moisture deficit
SOIL	soil index, being a weighted sum of SOIL1, SOIL2, ...SOIL5
SOIL1, 2, etc	fraction of soil in class 1, 2, etc
STMFRQ	stream frequency (junctions/km^2)
T	return period, average interval between years with a flood exceeding a given magnitude
TB	baselength of simplified unit hydrograph (hours)
T_p	time to peak of T hour unit hydrograph, measured from start of response runoff
URBAN	fraction of catchment in urban development
Y,y	reduced variate, used instead of probability to provide a linear relation with Q for a particular distribution
y_i, i = 1, 2, ...N	reduced variate values assigned to lowest, second lowest and so on of a series of observations.
α	scale parameter in extreme value distributions
β	scale parameter of exponential distribution
λ	average number of peaks exceeding a threshold, q_o, per year
μ	the mean of any distribution
σ	standard deviation